LAND SUBSIDENCE

Edited by

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Proceedings of the Third International Symposium on Land Subsidence held in Venice, Italy, 19-25 March 1984; the symposium was sponsored by IAHS, UNESCO, the Italian National Research Council (CNR), and some Italian regional and municipal authorities

IAHS
AISH
IAHS Publication No. 151

Published by the International Association of
Hydrological Sciences 1986.
IAHS, Institute of Hydrology, Wallingford,
Oxfordshire OX10 8BB, UK
IAHS Publication No. 151.
ISBN 0-947571-40-X

ACKNOWLEDGEMENTS
The success of the Symposium and of the Proceedings is the result of dedicated efforts by many people. The efforts of the Program Organizing Committee, consisting of representatives of all sponsoring and cooperating organizations, is greatly appreciated. All papers were reviewed by several reviewers for scientific and technical merit, and their assistance is especially appreciated. The organizations sponsoring the symposium along with IAHS--the Italian National Research Council (CNR); Italian Municipalities of Venice, Ravenna, and Modena; Italian Regions of Veneto and Emilia-Romagna; and the Venice Province, and the cooperating organizations--United Nations Educational, Scientific, and Cultural Organization (UNESCO); International Association of Hydrogeologists (IAH); International Society for Soil Mechanics and Foundation Engineering (ISSMFE); and the Association of Geoscientists for International Development (AGID), were helpful in financial as well as many other ways and their assistance is gratefully acknowledged. Finally, the Proceedings would not have been possible without the assistance of the IAHS Editorial Office, particularly Penny Kisby, and the Publications Office of the American Geophysical Union, particularly Bonny Gaston.

The camera-ready copy for this publication was prepared by the authors, and assembled papers were prepared by the American Geophysical Union.
Printed in the United States of America.

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Preface

The Third International Symposium on Land Subsidence was held 19-25 March 1984 in Venice, Italy--a site of major subsidence-induced problems. The symposium was sponsored by the Ground Water Commission of the International Association of Hydrological Sciences (IAHS); Italian National Research Council (CNR); Italian Municipalities of Venice, Ravenna, and Modena; Italian Regions of Veneto and Emilia-Romagna; and the Venice Province. Cooperating organizations included the United Nations Educational, Scientific, and Cultural Organization (UNESCO); International Association of Hydrogeologists (IAH); International Society for Soil Mechanics and Foundation Engineering; and the Association of Geoscientists for International Development (AGID). The papers published in this proceedings represent selected papers presented orally or by poster at that symposium.

The problems of land subsidence were among those included in the list of research projects needed during UNESCO's International Hydrological Decade, which began in 1965, and the International Hydrological Program--a continuation program beginning in 1975. These stated research needs resulted in the IAHS sponsorship of the International Symposium on Land Subsidence in 1969 in Tokyo, Japan, the Second International Symposium on Land Subsidence in 1976 in Anaheim, California, USA, and the Third International Symposium held in 1984 in Venice Italy. All three symposia have been held in locations of major subsidence problems. A Fourth International Symposium is being planned for 1989 at some other subsiding area. Papers from the first symposium were published as IAHS Publications No. 88 and 89 and papers from the second symposium were published as IAHS Publication No. 121.

The purpose of the Third (Venice) Symposium was to bring together international interdisciplinary specialists in the problems of land subsidence and to present results of research and practice in the subject; to exchange with all participants the experiences related to cause, effect, and control of subsidence and to the remedial works; and to provide a forum for discussion of the legal, socio-environmental, and economic impact of subsidence. This symposium was somewhat different from the others in several aspects. One is that it was broader, more interdisciplinary in coverage. A second aspect was the ability to finally provide results of some remedial measures and of environmental and economic consequences that normally take a decade or two to develop. The program showed the potential inter-relationships of subsidence characteristics, methods of study, and means of remedial work. The need for a broad interdisciplinary approach to any study of subsidence and to correction of resultant problems was another aspect adequately demonstrated by the program and by the wide range of sponsors and cosponsors that worked together to develop that program.

The symposium was held at the beautiful Cini Foundation on the historical Isle of San Giorgio Maggiore, Venice, Italy. The Foundation is a 1,000 year old Benedictine monastery with meeting rooms still filled with paintings, tapestries, and furniture of the great Italian artists and artisans dating as far back as the XVth century. Two evenings had special social events--a concert of classic Venetian music in an old church and a special guided tour of an art museum now occupying a former palace. Papers sessions occupied four days of the week. In mid-week, nearly 200 people took a one-day boat tour in the Venice Lagoon including places where subsidence occurred due to water extraction, the island of Poveglia where upheaval had been created by pressure grouting, to other islands--interesting for their glass and lace industries, and to tidal-control construction sites. On the weekend following the papers sessions an optional two-day field trip by bus took nearly 100 people to points of subsidence, hydrogeological and historical interest in areas from Venice through the Po Delta and the Emilia-Romagna littoral to Ravenna and Modena and return to Venice. Excellent guide books were available for the one-day and two-day field trips.

Over 200 attendees from about 30 countries not only received useful information from the nearly 100 oral and poster papers, but also enjoyed the pleasures of Italian food, hospitality, and good weather. Presented orally were fourteen papers on subsidence theory and modeling, five on instrumentation and measurement, ten on subsidence case histories related to ground--water withdrawal in six countries (China, Italy, Japan, Netherlands, Thailand, and USA); eight on case histories related to oil, gas, and mineral extraction, six on karst "sinkhole" type subsidence, five related to hydrocompaction and subsidence of organic deposits, three on remedial measures, and ten related to legal, socio-economic, and environmental aspects. Six papers also presented a detailed case history on the subsidence project for the Markerwaard, Netherlands, during which session papers covered all aspects of a study from original planning through geologic, hydrologic, and geotechnical studies to methods for damage estimates for buildings and development of remedial measures. Because of time constraints, 26 papers were presented by poster sessions. Some of these authors submitted full-length papers after the symposium. Following review and any needed revision 85 of the papers have been published in this proceedings directly from author's prepared copy. Opening remarks also are included in this volume and Co-chairman Johnson's closing remarks are incorporated in this preface.

General-Co-chairmen for the symposium were A. Ivan Johnson, Consulting Engineer, Arvada, Colorado, USA, and Lucio Ubertini, National Research Council's Institute for Hydrologic Research, Perguia, Italy. Laura Carbognin, National Research Council's Institute for Study of Dynamics of Large Masses, Venice, Italy, was chairperson for local arrangements, and was assisted by Francesco Marabini, Institute of Marine Geology, CNR, Bologna, Italy. The Technical Program Committee was chaired by Ivan Johnson, and Joseph F. Poland, Chairman Emeritus, U.S. Geological Survey, Sacramento, California, USA. Other members of the committee included Vincenzo Cotecchia, CNR Representative, University of Bari, Italy; Soki Yamamoto, IAHS Representative, University of Rissho, Tokyo, Japan; Pierro Sembenelli, ISSMFE Representative, Consultant, Milan, Italy; Pablo Girault, Consultant, ISSMFE Representative, Mexico City, Mexico; Tommaso Gazzolo, IHP Representative, Rome, Italy; John S. Gladwell, UNESCO Representative, Paris, France; Aurelio Aureli, IAH Representative, University of Catania, Italy; Robert T. Bean, IAHS Representative, Consulting Geologist, La Crescenta, California, USA; Prinya Nutalaya, AGID Representative, Asian Institute of Technology, Bangkok, Thailand; Laura Carbognin, CNR, Venice, Italy; German Figueroa Vega, Water Commission Valley of Mexico, Mexico City, Mexico; Francis S. Riley, U.S. Geological Survey, Denver, Colorado, USA; and H.L. Koning, Laboratory for Soil Mechanics, Delft, The Netherlands.

On behalf of the Symposium Organizing Committee, we express sincere gratitude to the cosponsors and cooperating organizations for their financial or services support to the symposium. Also acknowledged with appreciation are the contributions of the session chairmen, the authors, and the many people at visited towns and at the Cini Foundation, who gave of their time, effort, and knowledge to produce the successful program and field trips. A special thanks goes to Jane Zanin and Betty Johnson who provided the very heavy typing assistance needed for the symposium. Also a special thank you to Ruth da Costa, Sabina Gatto, and Alberta da Santi who kept work flowing smoothly at the registration desk.

It is hoped that the interdisciplinary nature of the program and of the sponsorship of this symposium and its proceedings will encourage multidisciplinary efforts to solve the many problems related to land subsidence. May all readers find this publication interesting and informative.

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WELCOMING REMARKS

Gaetano Zorzetto, Deputy Mayor
Environmental Department
City of Venice, Italy

General co-chairmen, technical committee members, ladies and gentlemen, I welcome you to our city as participants in this Symposium on Land Subsidence.

Venice, whose ground level is about 80 cm above the level of the lagoon water, is one of the sites where subsidence induced serious consequences on the environment. In fact during the present century Venice sunk about 25 cm in the lagoon. Almost half of these 25 cm must be ascribed to the heedlessness of man. He indeed performed a rash exploitation of underground water for industrial purposes as the best bargain. It is useless to quantify the problems, the costs and the risks suffered by this unique city as a consequence of an avoidable but now irrecoverable subsidence. The enormity of such problems, costs and risks can be easily realized.

The year 1984 will be very important for Venice. After ten years of studies, discussions and special laws, a plan of a total intervention for reclaiming and preserving the lagoon ecosystem, the defense against floodings, and the restoring of city buildings will be defined.

This requires an enormous financial commitment--two million millions lire in ten years--which will be supported by the local and national governments. For this reason they need to count on a profound scientific and technological knowledge.

The choice of Venice as the place to hold the Third International Symposium on Land Subsidence assumes then an emblematic value for human intervention in safeguarding its own vital environment.

It is an honor for the Municipality of Venice to have had the opportunity of collaborating with the organizing committee, and especially with the incomparable Laura Carbognin, in hosting this symposium at the magnificent Cini Foundation.

I welcome the participants on behalf of the city and I sincerely wish a productive and successful meeting, the rewards of which I hope will be beneficial to Venice.

WELCOMING REMARKS

Lucio Ubertini
Institute of Hydrologic Research
National Research Council
Perugia, Italy

Italian National Representative, IAHS

Good morning ladies and gentlemen. It is my great pleasure to greet you on behalf of the Italian Hydrological Community. First of all I should like to thank all those who have contributed towards the organization of this symposium, in particular the General Co-chairman, Dr. Johnson and the Chairman of Local Arrangement, Dr. Carbognin. They are to be commended for the way in which they have undertaken much of the work involved in the general organization, right down to the smallest details. Special thanks go, too, to the various organizations which have shown sympathy towards this initiative by giving us financial support. Finally, a heartfelt thanks to all those authors who responded to our requests for the presentation of their studies.

The great number of papers presented at this symposium is a further demonstration that research in the hydrological field springs from concrete problems. The planning and development of such research is sharply focused whenever the results obtained can contribute to resolution of problems of common interest. The hydrologists' contributions may be aimed at improving living conditions, as when, for example, they are concerned with the optimum use of soil or of surface or underground water resources, but they may also be aimed at conserving property and the natural environment and, sometimes, even at saving human lives. This may come about when the aim is to obtain results which permit us to combat such adversities as floods, landslides, soil erosion, land subsidence, and so on.

And it is precisely in this spirit that I consider the decision to hold the Third International Symposium on Land Subsidence here in Venice as being very apt indeed. Venice, like other cities in the same region, and Ravenna, like other cities in nearby regions, are suffering grave problems connected with subsidence phenomena.

These two cities merit, and need, particular attention on the part of scientists and engineers because of the unique position the cities occupy in the world's historic, artistic and cultural heritage. It is, in fact, necessary to take action as soon as possible to avert the danger of worsening subsidence that would cause this priceless artistic patrimony--already severely tried by years of this phenomenon--to suffer further deterioration. I believe there are very few other places in the world where a further slight subsidence would cause such serious damage as would result here in Venice. If the land continues to subside at the same rate as in recent years, the already disastrous situation caused by such phenomena as flooding from high tides would worsen considerably.

I should like to take this opportunity to briefly outline a few facts regarding the Italian hydrological community, taken from "The Italian National Report to IAHS" which was presented at the Eighteenth General Assembly of the IUGG in Hamburg last year. In the period from 1980 to 1983 in Italy, scientific activity in the hydrological field was carried out by 52 organizations, including 37 University Departments and 7 National Research Council Institutes. In the same period 283 scientific studies of an international character were made, of which a large number were concerned with subsidence phenomena.

Numerous conventions, symposiums and seminars have been organized in Italy. Particular attention has been given, and is still being given, to the developing countries--with 3 advanced courses in hydrology and water resources management being organized every year. Last year the very active Italian IAHS Committee was organized whose main aim is to promote the presence of Italian hydrologists on the international scene.

Therefore, it is with enthusiasm and gratitude that we welcome you here today to the opening of this symposium. The hydrological community looks at this as an opportunity to confront and verify studies and research that are being carried out all over the world and also, to hope that this marvelous city may benefit in some way.

Thank you for you attention, and may your work be fruitful.

WELCOMING REMARKS

Vincenzo Cotecchia
University of Bari
Bari, Italy

Representative of the National Research Council

Mr. Mayor, authorities, general co-chairmen, colleagues, ladies and gentlemen, it is an honor for me to represent the National Research Council of Italy (CNR) at the Third International Symposium on Land Subsidence. On behalf of the CNR President and myself, I cordially welcome all of you with the hope that this meeting will bring about scientific results. I hope that you will enjoy your stay here in Venice and that the weather will be warm like the heart of this city.

I participated in the two previous symposia of Tokyo and Anaheim and I had the opportunity of noticing both how relevant and wide the topics discussed were and how deeply the practical and scientific approaches were dealt with. I have now to consider how and how much the interdisciplinary methods of studying land subsidence have enlarged with respect to the previous editions.

The CNR is glad to be among the sponsors of this symposium for many reasons. The first of them is the interest that the phenomenon of land subsidence induces in the Italian territory; in fact, it was one of the fundamental topics of the CNR special project called "Conservazione del suolo" (Land Conservation) which recently ended after six years of activity.

The cases of land sinking in the Po River Delta, Bologna, Pisa, Modena, and particularly Venice and Ravenna, caused by the removal of fluid from the subsoil, are well-known. But unfortunately these are not the only phenomena: cases develop with a variety of features and causes, ranging from the acceleration of natural compaction of some areas to those suddenly induced by quarries, karst, to those devastating aspects sometimes slow, sometimes sudden related to the magma evolution in shallow depth up to real earthquakes.

Careful attention has been devoted by CNR to these dramatic and stimulating topics as evidenced by the many Italian papers presented at this symposium.

Last but not least, the fundamental reason testifying the massive participation of the CNR in this meeting is that right here in Venice some researchers of CNR from Istituto Studio Dinamica Grandi Masse, first begun successfully in Italy, systematic studies on the sinking phenomena of Venice and the Po Valley, carrying out highly qualified research leading to results well-known among the international scientific community.

At this point I want to recall one of the scientists of the above mentioned institute: Dr. Paolo Gatto, who recently died prematurely. His intelligent and untiring scientific activity in the field of land subsidence is in the final report of the closing session of the special project "Conservazione del Suolo", as well as in the posthumous papers presented in this symposium.

I have also to mention the attentive activity of CNR and its researchers in disseminating knowledge of the problems related to land subsidence also among non-technical people. Scientists who really feel the meaning and the gravity of the environmental situations revealed by their studies, have to play the role of sensible citizens of their country if they care about the results of their work. In fact, they link their scientific activity to that of disseminating knowledge and results. And so I am very pleased to realize that this meeting has been enthusiastically sponsored not only by the scientific organizations but also by political and local ones in the common goal of reducing the damage that land subsidence produces on the territory.

On this occasion we urge political and local authorities of our country to work hard together on the study of organic laws relative to the protection of the territory. This cooperation will undoubtedly arise on careful application of all natural resources, a strict control of human activities as a safeguard from catastrophic events.

The large participation in number of both qualified papers/posters presentations and experts are certainly the best base for the success of this symposium. The organization is excellent. Let me give my particular thanks to Drs. Ivan Johnson and Laura Carbognin for the untiring and passionate work done.

Concluding my opening remarks, I am sure that CNR will give the proper evaluation to the results derived from this meeting. Moreover, in the role of the scientific counterpart in dealing with political authorities, CNR will not only properly transfer the results accomplished, but also intensify studies, research and surveys and promote cooperation at all levels.

WELCOMING REMARKS

Laura Carbognin, CNR
Istituto Studio Dinamica Grandi Masse
Venezia, Italy

Chairman for Local Arrangements

General Co-Chairmen, Technical Committee members, ladies and gentlemen, we are very pleased to welcome you to this meeting.

Let me say that Venice is probably one of the most appropriate places to hold a symposium on land subsidence because of the worldwide interest in its sinking. This interest is due not to the magnitude of the subsidence but to the particular site where it took place and for the concern it created above all in terms of floodings.

By the way, Professor Vincenzo Cotecchia on behalf of CNR and Mr. Gaetano Zorzetto as deputy mayor of Venice have already appropriately dealt with the choice of Venice as the site for this third edition of the symposium.

I don't think it is necessary to recall either the scientific importance or the fundamental issues of such a meeting; the presence here of scientists and experts from twenty-five countries and the information and knowledge provided by the authors of the more than ninety papers and posters speak for themselves. I warmly welcome all of you; I wish you a successful and productive meeting and enjoyment of the city and the scheduled field trips.

I wish also to thank the sponsors and co-sponsors of this symposium among whom in particular the following local organizations:
The Municipality of Venice for hosting the meeting at the magnificent Cini Foundation; the Province of Venice, the Municipality of Ravenna, the Veneto and Emilia-Romagna Regions for their financial support and help during the field trips; the Municipality of Modena which printed the guidebooks and will give us a special hospitality during the two-day field trip; and finally the Province of Ferrara which will kindly host us at the Mesola Castle during the trip.

My welcoming address to you has a somewhat deeper meaning today not only because I live here and I have locally organized this meeting, but because it is on this occasion that I want to recall the memory of my colleague and friend Paolo Gatto, with whom I started organizing this symposium and who unfortunately and suddenly passed away last June. In dedicating this meeting to him I prefer to express myself in Italian not only because it is more spontaneous and more expressive for me, but also as a tribute to him, since Paolo was not very familiar with English and I'm sure that he would appreciate this token of personal friendship from all of us.

Ricordare e commemorare brevemente l'amico e collega Paolo Gatto non è un compito facile. Infatti per quanti hanno avuto l'opportunita di conoscerlo è evidente l'impossibilità di poter parlare di lui con poche frasi, ognuna delle quali sembrera insufficiente a delinearne la figura sia da un punto di vista scientifico che umano. E d'altra parte chi non lo conosceva non potra comprenderne il valore ne valutarne

It is not easy to commemorate in a few words Paolo Gatto, our colleague and friend. Indeed, for all those who had the opportunity of knowing him it is clearly impossible to speak of him with just a few sentences, each of which seems insufficient and limiting from both a scientific and human point of view. On the other hand, those who did not know him will not be able to understand his value and evaluate la grande perdita attraverso parole comunque inadegaute a ricordarlo.

Sotto l'aspetto scientifico la formazione di Paolo e avvenuta presso l'Universita di Padova dove nel 1963 si e laureato in Scienze Geologiche con il massimo dei voti e lode e dove ha trascorso i primi anni sucessivi la laurea dedicandosi sia all'attività didattica, sia all'attività di ricerca nel campo della geologia struttruale e stratigrafica delle Alpi, fornendo numerosi contributi scientifici. Nel 1969 entro come ricercatore nel CNR presso l'Istituto per lo Studio della Dinamica delle Grandi Masse di Venezia. Qui diede l'avvio a studi articolati e completi sulla subsidenza della pianura Veneto-Romagnola con particolare riguardo alle aree di Venezia e Ravenna.

Egli fu sempre sostenitore della necessita di studi interdisciplinari; e su questa base mise a punto agili metodologie di indagine, oggi comunemente adottate in questo tipo di ricerca. Egli inoltre promosse contatti tra studiosi di discipline diverse che difficilmente senza il suo intervento avrebbero avviato studi e ricerche in collaborazione.

Parte della sua attivita scientifica fu inoltre rivolta a studi stratigrafici, idrogeologici e paleomorfologici rigaurdanti la laguna di Venezia, della quale ha precisato l'origine, l'evoluzione naturale e quella indotta dall'uomo.

Lo stesso dipartimento ambiente del Comune di Venezia lo ha spesso chiamato a studiare problematiche sulla laguna e sulla citta di Venezia, da lui tanto amata. Inoltre come e-sperto del settore faceva parte del Gruppo UNESCO per gli studi ambienta li su Venezia.

Alle sue indubbie doti di acuto ed intelligente ricercatore univa quelle di onestà, generosità ed umanità nel lavoro e nella vita. Doti che hanno fatto di lui una persona amata e stimata da quanti lo hanno conosciuto. La sua improvvisa scomparsa ha privato il mondo/

the great loss through simple words, however inadequate in remembering him.

In the scientific aspect, Paolo Gatto's formation began at Padua University where he received his degree summa cum laude in geological sciences in 1963 and where he spent his first professional years teaching and doing research in structural geology and stratigraphics of the Alps. In 1969 he joined CNR as a researcher at the "Istituto per lo Studio della Dinamica delle Grandi Masse", in Venice, where he set in motion manifold studies on land subsidence of the Po Plain with particular regard to the areas of Venice and Ravenna.

He always supported interdisciplinary studies and with this in mind perfected research methods which are today commonly used in this field. He promoted contacts between experts in different fields, who, without his contribution, would, only with great difficulty, have started to collaborate on studies and research.

Part of his scientific activity was directed towards stratigraphical, hydrogeological and paleomorphological studies concerning the Venice Lagoon detailing its origin, natural evolution and man-induced trends.

The environmental department of the Municipality of Venice often called on him to study the problems of the lagoon and the city of Venice that he loved so much.

As an expert in this sector he was part of the UNESCO group of Venice environmental studies. Quite apart from his undoubted talent as a perceptive, intelligent researcher, his honesty, generosity and humanity both in work and in life were outstanding, making him beloved and esteemed by all who knew him. His sudden death deprived the scientific community of a precious researcher and took from his friends an irreplaceable person. In his dedication to his scientifico di un prezioso ricercatore, e ha tolto alle persone a lui vicine una presenza insostituibile.

Nella sua dedizione al lavoro che svolgeva con passione egli aveva una enorme carica di generosita verso i colleghi tra i quali combatteva invidie e rancori favorendo il rispetto reciproco come regola di convivenza.

Si può dire che nel mio Istituto e negli altri ambienti che egli frequentava non ci sia persona che in un modo o in un altro non abbia ricevuto molto da Paolo Gatto e non serbi di lui un ricordo più che caro.

Nel ricordo ricorrente, tra colleghi, emergono aneddoti o frasi che hanno lasciato un segno. Valga per tutto cio la definizione che l'amico Franco Marabini ne ha dato recentemente chiamando Paolo Gatto "una persona rara". Chi l'ha conosciuto trova certo questo appellativo particolarmente adatto ad esprimere lo stato d'animo collettivo.

E davvero come persona rara lo ricorda chi vi parla, che ha collaborato con lui lungo un arco di tempo prossimo ai quindici anni: persona rara, come si è detto, per quanto ne proveniva di conoscenze scientifiche e vitù umane.

Ringraziamo quindi, nella memoria, Paolo Gatto e ringraziamo i presenti, anche quelli che non l'avevano conosciuto per la loro attenzione.

work, which he did with great passion, he was generous towards his colleagues, fighting envy and rancor, encouraging reciprocal respect as a role of living and working together. I can say that in my institute and in the other circles he frequented, there is not one person who in one way or another has not received a lot from Paolo Gatto and who does not have a very dear memory of him.

In the recurrent recollections of him among colleagues, anecdotes or phrases emerge which have left a mark. Our friend Franco Marabini recently called Paolo Gatto "a rare person"; those who knew him regard this definition as particularly apt at expressing the general state of mind. And it is as a "rare" person that I remember Paolo Gatto, having collaborated with him for nearly fifteen years; a rare person for both his scientific knowledge and human virtues.

Our thanks, then, in memory of Paolo Gatto, to all of those present even if they did not know him personally, for their kind attention.

OPENING REMARKS OF PROGRAM CHAIRMAN

A. Ivan Johnson
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Land subsidence, or land-surface sinking, has become a major man-induced hazard in many parts of the world--including here in Venice. Many of the subsiding areas have become known only in recent years, and have taken place primarily since World War II as a result of the rapidly increasing population and industry. Man's continuing development of ground water, gas, oil, and minerals has been changing the natural fluid regime in the subsurface at many locations throughout the world. Unfortunately, most planners of industrial complexes, urban developments, and water supply systems are not adequately informed about the potential hazards, costs, and the socio-economic and environmental problems that can result from land subsidence.

Subsidence can result from the compaction of sediments due to heavy withdrawal of water, oil, or gas and extraction of solids through mining, by the shrinkage and oxidation of organic deposits, the withdrawal of fluids for geothermal power, and the collapse of sediments contained in karst terrain. Many areas of subsidence are known, some having subsided as much as 10 m. Many more areas are likely to develop in the next few decades as a result of accelerated exploitation of natural resources, especially ground water, in order to meet the demands of the increasing population and industrial concentrations throughout the world.

Most areas of known subsidence are along coasts where the land sinking becomes quite obvious as the ocean or lake waters gradually creep higher and higher up the shore. The program for this symposium will show that flooding of populated and industrialized areas is a major problem resulting from subsidence of coastal areas at Tokyo, Japan; Houston, Texas, USA; Shanghai, China; Bangkok, Thailand; and Long Beach, California, USA. Estimates of the costs of damage or of remedial measures will be shown to be in the billions of dollars. Such subsided areas--Tokyo is an example--frequently must be protected from flooding as much as several meters by construction of extensive systems of dikes, flood walls, locks, and pumping stations. Other papers will show that changing gradients due to subsidence can seriously effect the capacity or drainage pattern of canals, drains, sewers, and streams. Structural failure of buildings, pipe lines, railroads, and other engineering structures at the land surface has occurred in some areas due to the slow tensional or compressional stresses caused by flexure of the sediments or due to the sudden, calamitous collapse of sink holes in karst areas. Compressional or shear failures of oil, gas, or water-well casings also occur frequently.

Subsidence due to ground-water withdrawal will be seen to range from about several hundred millimeters in Venice to about 10 meters in Mexico City, Mexico and the San Joaquin Valley, California to 15 meters in the Cheshire District of Great Britain, where rock salt has been mined by solution since Roman times. The areal extent of reported subsidence world wide is reported to range from 10 square kilometers in the San Jacinto Valley to 14,000 square kilometers in the San Joaquin Valley, both in California.

Other than with karst sink-hole conditions, subsidence usually is a subtle phenomenon. Despite the often large areal extent, the rate of subsidence may be relatively slow and so widespread that the problem is not readily evident until underground pipelines crack, well casings buckle, or shorelines are flooded. With modern technology, such as fluid injection to replace withdrawn water or oil, it is reported that subsidence can be slowed or stopped. However, it should be noted that subsidence is essentially permanent. There is no known method at present for raising the land surface back to its former elevation, although a small island in the lagoon of Venice has been raised above flood level by high pressure injection of fluid grout.

More information on subsidence is available in a new UNESCO publication. Dr. Joseph F. Poland chaired an UNESCO/IHP Working Group on Land Subsidence, which consisted additionally of Laura Carbognin of Italy, Soki Yamamoto of Japan, German Figueroa Vega of Mexico, and me from the USA. The working group's purpose was to put our collective information and knowledge together into a publication entitled "UNESCO Guidebook to Studies of Land Subsidence Due to Ground-Water Withdrawal." This 327-page volume can be ordered from UNESCO in Paris, or its publishing outlets in many countries.

In conclusion, our Symposium Organizing Committee hopes this Third International Symposium will provide a good forum for the sharing of useful ideas and experiences. It is fervently hoped that this symposium and the subsequent proceedings will bring to the attention of planners, developers, and politicians, as well as other scientists, what might be called "man's participation in a natural hazard--land subsidence."

I now open the technical paper sessions of the Third International Symposium on Land Subsidence. As the first step, I will read the following citation in honor of Dr. Joseph F. Poland, to whom this symposium and subsequent proceedings are dedicated, in recognition of his world-renowned work in land subsidence and his long role as the Father of Land Subsidence Hydrology.

Citation

Dr. Joseph F. Poland received his A.B. in geology from Harvard University, and M.A. in geology and Ph.D. in hydrogeology from Stanford University. For 10 years prior to joining the U.S. Geological Survey, Dr. Poland worked as a geologist with an oil company in South America, taught at Stanford University, and consulted in the southwestern United States on geologic and hydrologic problems related to ground-water development.

Dr. Poland began his distinguished career with the U.S. Geological Survey in 1940 as geologist assigned to complex investigations of geology, hydrology, and geochemistry of ground waters beneath the Los Angeles coastal plain. In 1946 he was placed in charge of the state-wide ground-water investigations and associated research programs in California. His leadership during that assignment led to the delineation of the major aquifer systems and their storage capacity which proved to be so essential to the development of the world's largest and most expensive water development project--the California Water Plan.

From 1956 to his recent retirement, Dr. Poland was responsible for planning and carrying out fundamental research related to land subsidence and associated studies in the mechanics of aquifer systems. This research led to the saving of millions of dollars of irrigation, aqueduct, and highway construction costs in areas of potential subsidence in California.

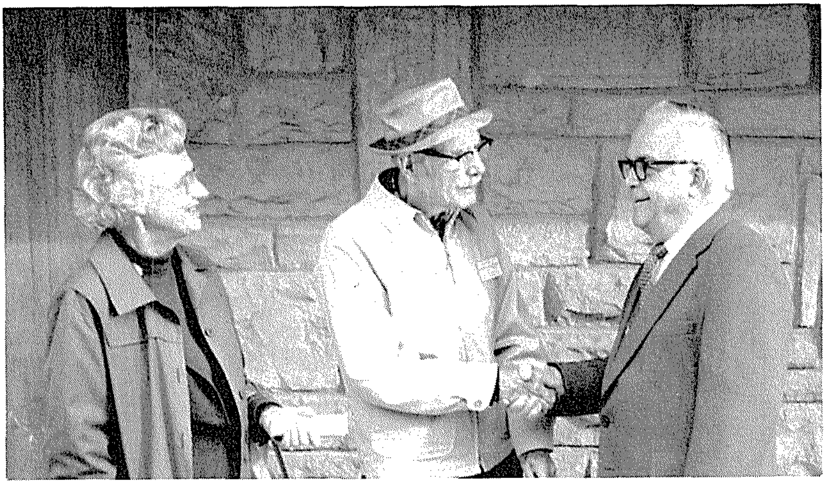
Dr. Poland developed an international reputation of renown in his field. He has been called in as an expert consultant in many parts of the U.S. and internationally by organizations such as UN, Unesco, and FAO. He served as the Chairman of the Unesco Working Group on Land Subsidence, which has produced the Unesco Guidebook to Studies of Land Subsidence Due to Ground--

water Withdrawal. He is a registered Geologist and certified Engineering Geologist in California. He has been active on many inter-agency committees and on organizing committees for international symposia and workshops. In 1968 he received the Department of the Interior's Distinguished Service Award for his outstanding scientific achievements, and in 1970 he received the Association of Engineering Geologist's Clair P. Holdredge Award for a paper considered an outstanding contribution to the profession of engineering geology.

Dr. Poland has authored more than 50 important scientific papers and reports on geohydrology, subsidence, geochemistry, mechanics of aquifers, and related aspects of hydrology.

By personal example, Dr. Poland has provided noteworthy leadership and has stimulated the flow of ideas and production of scientific contributions of co-workers. He particularly motivated the younger geologists and engineers with whom he worked during his 40-year career with the U.S. Geological Survey and is highly revered by such colleagues today, not only as an outstanding scientist but as a gentleman and a friend.

Dr. Poland's stature and reputation as a scientist, nationally and internationally, are outstanding. In recognition of his many achievements, the Proceedings of the Third International Symposium on Land Subsidence is dedicated to Dr. Joseph F. Poland.



Complimenting Dr. Joseph F. Poland (center) on his citation for outstanding achievement in land subsidence research is A. Ivan Johnson (right) Symposium General Co-chairman. Looking on with pleasure is Dr. Poland's wife, Eleanor (left).

ONE-DIMENSIONAL LAND SUBSIDENCE WITH VARIABLE TOTAL STRESS

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Abstract

The variation of total stress is observed in many cases where the overburden load decreases or increases due to changes on the ground surface or water table fluctuations. In this study, starting from three dimensional coupled equations of mass and equilibrium developed by Corapcioglu and Bear (1983) and introducing elastic stress-strain relations with variable total stress, we obtain a (vertical) one-dimensional model for surface settlement due to water table lowering in a vertical soil column. The model follows a set of solid particles, $F_2 = 0$, which at some initial time coincides with the phreatic surface, $F_h = 0$. Numerical results are presented graphically at the spatial and temporal points of interest for a value of the resistivity of the fixed semipervious lower boundary of the aquifer.

Introduction

The phenomenon known as land subsidence is the settlement of the land surface due to any of several factors among which the most important is fluid withdrawal. This withdrawal reduces the pressure in the pore water, that in turn increases the effective stress and thus causes compaction of the solid skeleton of the aquifer. In addition to land subsidence in confined aquifers, unconfined aquifers may also experience land subsidence due to lowering of water table. The loss of buoyancy of solid particles in the zone dewatered by the falling water table causes a decrease in the total (overburden) stress in the aquifer. Other examples of similar nature are the change of overburden load due to flooding of extensive areas and excavation and dewatering for construction purposes. Also, the fluctuations in water level in a reservoir would induce changes in total stress over the confined aquifer beneath. In estuaries and salt marshes, periodic tidal movements or the movements of sand dunes or surface infiltration would create changes in total stress which causes land subsidence.

Previous Studies

The variability of total stress in soil consolidation problems has been studied by several researchers in the literature. Gibson (1958) formulated a consolidation equation in terms of excess pore pressures in a soil which increases in thickness in time due to accreting soil layers. Koppula (1983) studied a similar problem in which, instead of addition of fresh soil layers, soil layers are removed at the top due to erosion and excavation and the sediment thickness decreases with time. It has been found that the deficient pore water pressure at any instant is a function of the ratio of the depth to the thickness of soil removed up to that instant and the ratio of the rate of soil removal to the swelling coefficient of soil. We should note that in the formulation the coefficient of consolidation is replaced by the coefficient of swelling. Baligh and Fuleihan (1978) studied a problem where the consolidation of deep clay deposits due to surface loading produces large surface settlements that cause significant changes in the stresses in the underlying clay and result in an unusual consolidation behaviour. They developed a one-dimensional theory of primary consolidation that accounts for the reduction in stresses caused by the settlement of a clay layer. Deviations from Terzaghi's theory due to changes in stresses caused by settlements are shown to increase with the compressibility of the clay layer and therefore become important in deep soft clay deposits. The resulting stress changes affect the final settlement, the excess pore pressures, and the rate of settlement.

Kashef and Chang (1976) developed a finite-difference numerical solution to determine the land subsidence due to groundwater pumping in unconfined aquifer overlying an aquitard. The changes in pore water pressure within the aquitard due to pumping from the upper unconfined aquifer are considered. The total land subsidence resulting from the compressiblity of both the aquifer and the aquitard is determined in the vertical direction by a time-dependent summation procedure based on the consideration of the pressure head diagram in the same direction. Madhav and Basak (1977) presented analytical solutions for rate of subsidence and pore pressure distribution in an aquitard caused by a sudden increase in the effective (intergranular) stress due to an instantaneous lowering of the water table in an overlying phreatic aquifer. They assumed a non-Darcy flow expansion of non-linear type. In a comparison of Darcian and non-Darcian flow, Madhav and Basak inadvertently concluded that subsidence and pore pressure distributions are independent of the thickness of the deforming medium and drawdown for Darcian flow. Safai and Pinder (1979) have considered three-dimensional land subsidence in an unconfined aquifer. The region above the water table was considered as an unsaturated porous medium. The equations governing saturated-unsaturated soil deformation are obtained as a simplification of multiphase formulation. Soil deformation due to flow of two immiscible fluids can be described by a general three-dimensional deformation field coupled with a three-dimensional flow field. The system of nonlinear partial differential equations governing saturated-unsaturated flow in a deforming porous medium was solved using an iterative Galerkin Finite Element technique. Safai and Pinder did not consider the variability of total stress in their formulation. Hemond et al. (1984) have formulated surface infiltration resulting from tidal flooding in a saturated compressible salt marsh. The effect of variable total stress has been included in a general equation in terms of piezometric head. Later, the resulting equation for the dynamic component of the piezometric head which varies only in response to the movement and storage of water in soil has been solved analytically.

Corapcioglu and Bear (1983) developed a mathematical model for the areal distribution of drawdown, land subsidence, and horizontal displacements due to groundwater pumping from a deformable phreatic aquifer. Following the development of a three-dimensional mathematical model consisting of a mass conservation equation, quasi-static equilibrium equations, and stress-strain relations for an assumed perfectly elastic solid matrix, a regional, horizontal two-dimensional model is derived by averaging the three-dimensional model over the vertical thickness of the aquifer, taking into account the continuous variation in total stress as a result of water table fluctuations. The separate movement of a fluctuating phreatic surface and that of a surface of solid particles, which initially coincides with the former, as well as the compaction of the aquifer, have been incorporated into the averaged model. An analytical solution is given for the case of a simple pumping well. Corapcioglu and Bear (1983) assumed that the change in total stress, referred to as excess total stress, at any point within the saturated domain is due to the change in overburden (soil and water) weight directly above that point. This approximation, referred to as the plane incremental total stress, is sufficiently good when the change in overburden weight is more or less uniform over the considered area. In principle, however, one may obtain this change by solving a boundary value problem for total stress which takes into account the changes in stress boundary conditions.

In this paper, the problem of land subsidence, drawdown, and pore pressure distribution with variable total stress caused by lowering of the water table in a vertical column of an aquifer, overlying an aquitard of small thickness is studied. An idealized definition sketch of the problem is given in Fig. 1.

Theory

We start with the *conservation of mass equation for water* in a saturated porous medium.

$$\nabla \cdot \underset{\sim}{q}_r{}^e + \frac{\partial \varepsilon^e}{\partial t} + n\beta \frac{\partial p^e}{\partial t} = 0 \qquad (1)$$

where $\underset{\sim}{q}_r{}^e$ is the specific excess discharge of the water relative to the moving solids, p^e is the pressure in the water (positive for compression), n is the medium's porosity, $\varepsilon = \nabla \cdot \underset{\sim}{U}$ is the volumetric strain of the soil matrix, $\underset{\sim}{U}$ is the displacement of the soil matrix, β is the coefficient of water compressibility, and t is time. Because we are interested only in subsidence, we separate the flow into a steady flow without subsidence, and an excess unsteady flow producing subsidence. In eq. (1), we denote the latter by supersrcipt e. The details of the derivation of eq. (1) and the other equations in this theoretical development are given in Bear and Corapcioglu (1983).

In the unconfined aquifer problem considered in this paper, the upper boundary of the saturated zone, that is the water table, and the upper boundary of the compacting soil do not always coincide. Therefore, we have to make a clear distinction between two surfaces. The first one is the phreatic surface, described by $F_h(z,t) = 0$, and the other is a set of points defining a surface, $F_2(z,t) = 0$ (see Fig. 2). If $F_1(z,t) = 0$ describes the semipervious lower boundary (e.g., an aquitard) then eq. (1) is valid in the region of thickness $B = F_1 - F_h$. Let the surfaces $F_h = 0$ and $F_2 = 0$ coincide at some initial time t. At some later time $t + \Delta t$, the surfaces are displaced, with the drawdown, s at the water table, the displacement $U_z|_{F_2}$ at the surface, $F_2 = 0$ which consists of a set of solid particles. If $F_1 - F_2 = D$ and the lower boundary $F_1 = 0$ is stationary, then $U_z|_{F_2} = D - D^o$. Here we consider a continuously declining water table and a compacting soil. We also assume that soil compaction is produced only within the saturated flow domain where changes in water pressure occur. Then it is transmitted upward,

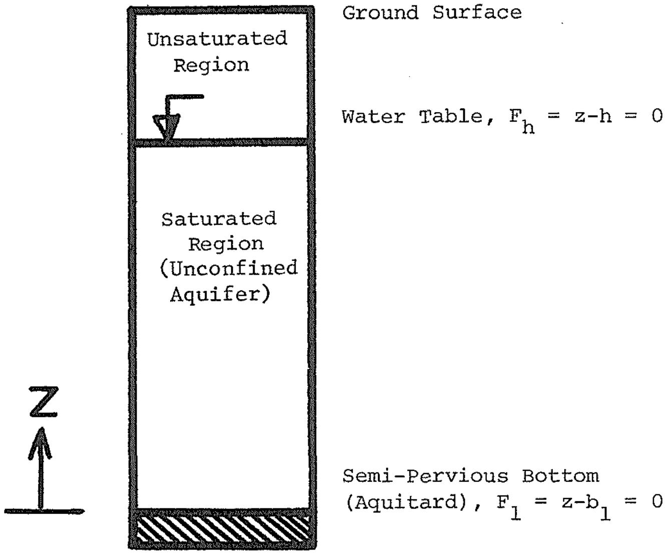

Figure 1. Schematic Diagram of One-dimensional Model.

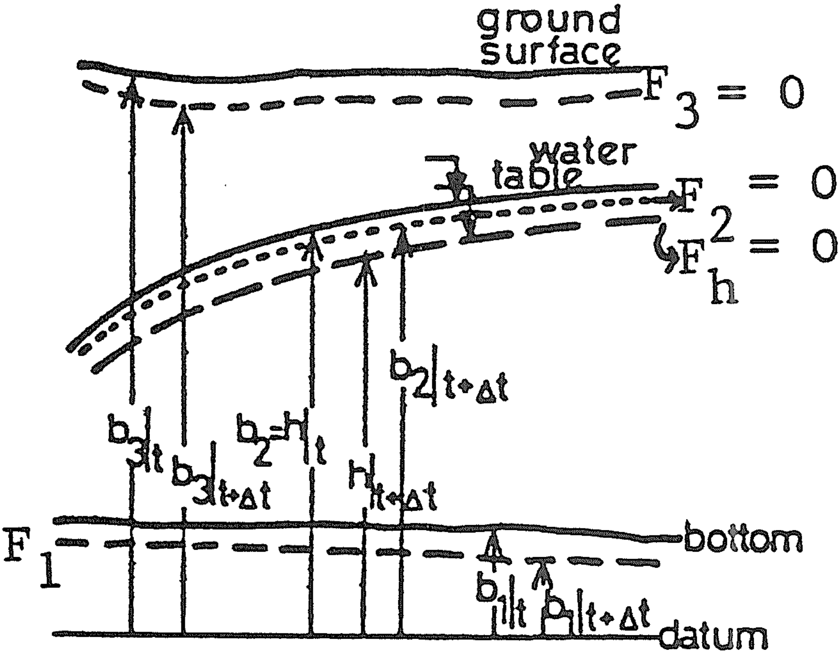

Figure 2. Chages in b_1, b_2, b_3, and h during a time step.

without change, to ground surface, where the land subsidence is observed. In other words, we assume that no consolidation takes place in the unsaturated zone above the water table. In reality, the displacement may be attenuated due to changes in the moisture content in this region.

Then, following the Terzaghi's concept of effective stress, we write

$$\underset{\approx}{\sigma} = \underset{\approx}{\sigma}' - p\underset{\approx}{I} \tag{2}$$

where $\underset{\approx}{\sigma}$ is the total stress, $\underset{\approx}{\sigma}'$ is the effective stress, and $\underset{\approx}{I}$ is the unit tensor. We assume that the soil matrix is isotropic and for the relatively small compaction here behaves as a perfectly elastic body, for which the stress-strain relationship is given by

$$\underset{\approx}{\sigma}'^{e} = G[\nabla \underset{\sim}{U} + (\nabla \underset{\sim}{U})^{T}] + \lambda \nabla \cdot \underset{\sim}{U} \tag{3}$$

where G and λ are the Lamé constants. The superscript T shows the transpose of a matrix.

We may now return to eq. (1), and with *vertical consolidation only*

$$\frac{\partial \varepsilon^{e}}{\partial t} = \alpha \frac{\partial \sigma'^{e}}{\partial t} = \alpha \frac{\partial}{\partial t} (\sigma^{e} + p^{e}) \tag{4}$$

where α is the coefficient of matrix compressibility , eq. (1) would yield

$$\nabla \cdot \underset{\sim}{q}_r{}^{e} + (\alpha + n\beta) \frac{\partial p^{e}}{\partial t} + \alpha \frac{\partial \sigma^{e}}{\partial t} = 0 \tag{5}$$

In eqs. (1) and (5), we express $\underset{\sim}{q}_r$ by Darcy's law

$$\underset{\sim}{q}_r{}^{e} = - \frac{\underset{\approx}{k}}{\mu} (\nabla p^{e} + \rho g \nabla z) \tag{6}$$

where $\underset{\approx}{k}$ is the medium's permeability, g is the gravitational acceleration, and z is the vertical coordinate. When there is no change in total stress, e.g., in confined aquifers, the last term of eq. (5) would vanish. In this study, we deal with *variable total stress*, that is, the change of excess total stress at any point within the saturated domain is related to the change in overburden weight of soil and water directly above that point due to the decline of the water table.

Lowering of the water table and associated compaction of the saturated zone will reduce the value of total stress. We assume that the change in excess total stress takes place only in the vertical direction. We also assume that the excess total stress at any point in the saturated zone between b_1 and h can be represented by the excess total stress at the midpoint, $\bar{z}$, of that zone. We further assume that the bottom surface b_1 remains fixed. These assumptions lead to

$$\sigma_z{}^{e} = \sigma_z{}^{e}\big|_{\bar{z}} = - \frac{s}{2}[n\gamma_w+(1-n)]\gamma_s] + (s-\Delta b_2)[\theta_o\gamma_w+(1-n)\gamma_s] \tag{7}$$

where $\Delta b_2 = b_2{}^{o} - b_2$, $s = h^{o} - h$. Superscript o denotes the initial configuration of the respective surfaces. s is the drawdown, γ_w and γ_s are the specific weights of water and soil grains, respectively, and θ_o is the irreducible moisture content of the soil above the water table. $\gamma_m{}^{*} = n\gamma_w+(1-n)\gamma_s$ is the specific weight of saturated soil. Note that $\Delta b_2 = -U_z\big|_{F_2}$. For $s >> \Delta b_2$, eq. (7) can be approximated by

$$\sigma_z^e = -s\left[-\frac{n\gamma_w + (1-n)\gamma_s}{2} + S_y\gamma_w\right] \tag{8}$$

where $S_y = n - \theta_o$ is the specific yield of the phreatic aquifer.

Boundary Conditions

For the phreatic surface, $F_h = 0$, which serves as the upper boundary of the saturated zone in the unconfined aquifer, based on the continuity of flux across the phreatic surface, Corapcioglu and Bear (1983) obtained

$$\underset{\sim}{q}_r^e\Big|_{F_h} \cdot \nabla F_h = -S_y \frac{\partial s}{\partial t} \tag{9}$$

where we neglect the rate of accretion which may introduce additional flux due to surface sources such as rainfall. We should note that on the phreatic surface, the pressure is always atmospheric, and hence

$$p\Big|_{F_h} = 0 \tag{10}$$

The lower boundary is a semipervious surface F_1, through which leakage takes place. This corresponds to an aquifer-aquitard system where the aquitard has a finite thickness. We assume that the aquitard behaves like a membrane with negligible compressibility. Furthermore, it is assumed that the flow in the aquitard is vertical. This assumption is valid when the permeability of the aquifer is much larger than that of the aquitard. Then, if this surface is a material surface with respect to soil particles, the boundary condition on this surface based on continuity of flux is

$$\underset{\sim}{q}_r^e\Big|_{F_1} \cdot \nabla F_1 = \frac{\left(\frac{p^e}{\gamma_w} + z\right)\Big|_{F_1+d'} - \left(\frac{p^e}{\gamma_w} + z\right)\Big|_{F_1}}{d'/K'} \tag{11}$$

where d' and K' are the thickness and hydraulic conductivity of the aquitard respectively. d'/K' is referred to as the resistivity of the membrane. For simplicity, let us take $p^e/\gamma_w|_{F_1+d'} = 0$. In reality, the pore pressure outside the aquitard would have a non-zero value. This assumption is justified in view of the fact that K' << K where K is the hydraulic conductivity of the aquifer. Then eq. (11) would yield

$$\frac{\partial p^e}{\partial z}\Big|_{F_1} = \frac{p^e|_{F_1}}{Kd'/K'} - \gamma_w(1 + \frac{K'}{K}) \tag{12}$$

We should also note that the initial pressure distribution is hydrostatic, i.e., $p(z,0) = \gamma_w(D^o - z)$.

One Dimensional Numerical Model

The governing equation (5) will be solved in a one-dimensional (vertical) flow field. The flow in the unconfined aquifer is only in the vertical direction due to the lowering of the water table. Accordingly, the displacement field $\underset{\sim}{U}$ has only the vertical component. Then, the substitution of eqs. (6), (8), and (9) into eq. (5) would yield an equation only in terms of excess pore pressure. Eq. (9) is employed to express drawdown in terms of vertical pore pressure gradient at water table. Then

$$- C_v \frac{\partial^2 p^e}{\partial z^2} + \frac{\partial p^e}{\partial t} - J \left.\frac{\partial p^e}{\partial z}\right|_{F_h} = J \gamma_w \Big|_{F_h} \tag{13}$$

where $C_v = K/[\gamma_w(\alpha+n\beta)]$ and $J = -K\alpha\{([n\gamma_w+(1-n)\gamma_s]/(2S_y\gamma_w))-1\}/(\alpha+n\beta)$. The constant C_v is known as the coefficient of consolidation in soil mechanics. The solution of eq. (13) subject to boundary conditions given by eqs. (10) and (12) would give estimates of the pore pressure. To achieve this, we employed the Galerkin finite element method. The method involves the approximation of p^e with

$$p^e \approx \sum_{i=1}^{n} N_i(z)\ P_i(t) \tag{14}$$

where $N_i(z)$ are known interpolation functions. After the application of the Galerkin techniques, eq. (13) would give

$$- C_v \frac{dP_i}{dz} N_j \Big|_{F_1}^{F_h} + \frac{d}{dt} P_i < N_i, N_j > + C_v P_i < \frac{dN_i}{dz}, \frac{dN_j}{dz} >$$
$$- J \left.\frac{dP_i}{dz}\right|_{F_h} < N_j > = J \gamma_w \Big|_{F_h} < N_j > \tag{15}$$

With an implicit scheme for time derivative, eq. (15) becomes

$$- C_v \frac{dP_i}{dz} N_j \Big|_{F_1}^{F_h} + P_i^{t+\Delta t} \left[C_v < \frac{dN_i}{dz}, \frac{dN_j}{dz} > + \frac{1}{\Delta t} < N_i, N_j > \right] - J \left.\frac{dP_i}{dz}\right|_{F_h} < N_j >$$
$$= \frac{P_i^t}{\Delta t} < N_i, N_j > + J \gamma_w \Big|_{F_h} < N_j > \tag{16}$$

where Δt is the time step. Once the pore pressure values are calculated, we can obtain the temporal change of drawdown by eqs. (9) and (6). In one-dimensional form

$$\Delta s = \frac{K}{S_y} \frac{\Delta t}{\gamma_w} \left[\left.\frac{dp^e}{dz}\right|_{F_h} + \gamma_w \right] \tag{17}$$

where Δs is the incremental drawdown. To calculate s at time $t+\Delta t$

$$s^{t+\Delta t} = s^t + \Delta s \tag{18}$$

To obtain a one-dimensional expression for Δb_2, we write $\sigma_{zz}'^e$ from eq. (3), substitute it to eq. (2), employ eq. (8)

$$- s \left[- \frac{\gamma_m^*}{2} + S_y \gamma_w \right] = (\lambda + 2G) \frac{\partial U_z}{\partial z} + \frac{s}{2} \gamma \tag{19}$$

Note that the relationship between the change in pore pressure p^e, within the flow domain, and change in water table elevation s is expressed as

$$p^e = - \gamma_w s/2 \tag{20}$$

which was obtained by Corapcioglu and Bear (1983). Also, for a one dimensional deformation and flow field, the coefficient of matrix compressibility $\alpha = 1/(\lambda+2G)$. Integration of $\partial U_z/\partial z$ from b_1 to b_2 would give

$$\Delta b_2 = - U_z\Big|_{F_2} = - s\left[- \frac{n\gamma_w + (1-n)\gamma_s}{2} + \frac{\gamma_w}{2} + S_y\gamma_w\right]\alpha \; D \tag{21}$$

The fact that the two surfaces - the phreatic surface and that of an initial set of solid particles, the displacement of which is being followed are different ones causes an inherent difficulty in the solution. In the numerical technique, we redefine after each time interval, the new F_2 surface to coincide with the new F_h surface (i.e., D = B), so that the two surfaces follow each other except for their separation during a single time interval Δt. At every time step, we solve (16), (17), and (21) for pore pressure p^e, drawdown s, and surface settlement Δb_2. Then we redefine $D^{t+\Delta t} = D^t - (s-\Delta b_2)$ and solve for the next time interval. This routine stems from our assumption that no compaction occurs above the water table.

Results

The numerical approach described to solve the governing equations (16), (17), and (21) is applied to a 1-m-high soil column packed with sand with a membrane at the bottom. This application illustrates a phreatic aquifer-aquitard system. Initially the whole thickness of the soil column is assumed to be saturated, i.e., the water table coincides with the ground surface. For purposes of numerical simulation the column is divided into 10 volume elements of equal size. During each time step the size of volume elements has been adjusted by considering the change in the height of saturated column due to drawdown and displacement. The time step Δt is taken as 60 seconds. The results are presented graphically in Figs. 3 and 4. The model parameters used in the numerical simulation are given in Table 1.

As seen in Fig. 3 the soil column is almost completely drained after 18 days. Note that the pressure distribution remained hydrostatic with time after undergoing gravity drainage. The thin porous membrane offers some resistance to the flow of water through it. For lower membrane permeability, the resistance to the flow increases, resulting in larger pore pressures throughout the column. If the permeability of the membrane is infinitely small, the pore pressure distribution along the column becomes parallel to the initial hydrostatic condition at all time steps. A smaller value of Kd'/K' would give pore pressure values at the bottom much less than the hydrostatic pressure. The column experienced 90% of its maximum consolidation after 8 days and after that, remained almost constant with time. Likewise the rate of change of drawdown and total stress is large at earlier times until it attains 90% of its maximum value after 8 days, after which it tends to become asymptotic with time.

The simulations described in this study strongly suggest that in phreatic aquifers the importance of drawdown on land subsidence cannot be ignored although the magnitude of excess total stress is quite small in comparison to excess pore pressure.

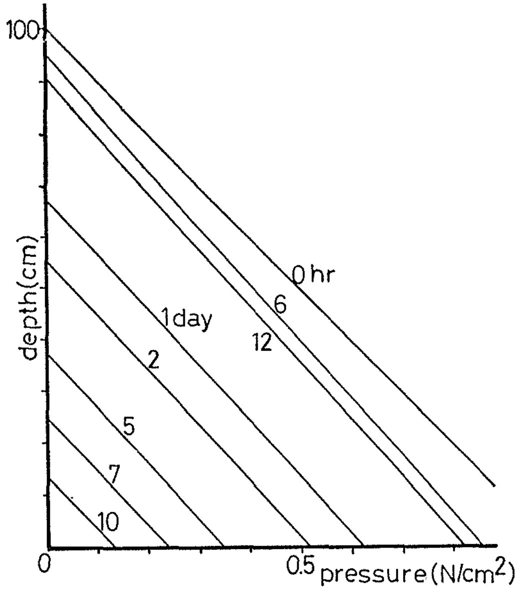

Figure 3. Spatial and Temporal Change of Pore Pressure for Kd'/K'=1,000 cm.

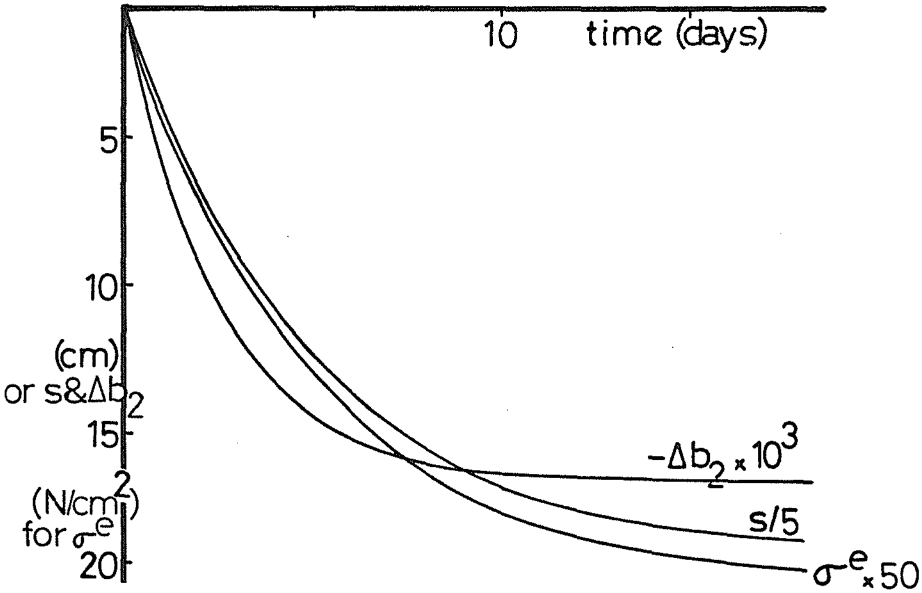

Figure 4. Temporal Change of Subsidence, Drawdown, and Excess Total Stress.

Table 1. Model Parameters

Hydraulic Conductivity of Fine Sand Column	$K = 1 \times 10^{-3}$ cm/sec
Hydraulic Conductivity of Clay Membrane	$K' = 1 \times 10^{-6}$ cm/sec
The Coefficient	$Kd'/K' = 1{,}000$ cm
Compressibility of Soil	$\alpha = 5 \times 10^{-3}$ cm^2/N
Compressibility of Water	$\beta = 5 \times 10^{-6}$ cm^2/N
Viscosity of Water	$\mu = 8 \times 10^{-8}$ N.sec/cm^2
Specific Weight of Water	$\gamma_w = 9.81 \times 10^{-3}$ N/cm^3
Density of Soil Grains	$\rho_s = 2.1$ g/cm^3
Density of Water	$\rho = 1.0$ g/cm^3
Specific Weight of Saturated Soil	$\gamma_m{}^* = 16.285 \times 10^{-3}$ N/cm^3
Porosity	$n = 0.4$
Specific Yield	$S_y = 0.4$

Acknowledgement

The first and third authors appreciate the Engineering Foundation Grant RC-A-79-1A for partial support of this work. The second author wishes to thank the Fund for the Promotion of Research at the Technion for providing partial support for the research leading to the present paper.

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LAND SUBSIDENCE DUE TO GAS-OIL REMOVAL IN LAYERED ANISOTROPIC SOILS BY A FINITE ELEMENT MODEL

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Abstract

A parametric analysis of land subsidence due gas-oil withdrawal from a reservoir overlain by layered transversally anisotropic soils has been performed by a finite element model. A disk-shaped reservoir of uniform thickness has been assumed with a radius equal to 2500 m at an average depth of 1750 m.

The pressure decline has been set equal to 1 kg/cm^2 within the gas/oil bearing strata and has been assumed to vary linearly from 1 to 0 in the overlying and underlying low-permeable sediments.

The model assumes elastic deformations and allows for layered transversally anysotropic soil units each characterized by 5 distinct elastic constants. Above the reservoir a typical structure is made of alternating clays and sands with a compressibility which decreases progressively with depth. The compressibility values have been taken within the characteristic range of the Quaternary sediments of the Po river plain. The model was run for a number of possible stratigraphic configurations.

The results show that land subsidence depends primarily on the actual distribution of the elastic properties inside and around the reservoir and that the importance of an accurate soil description greatly decreases as one approaches the ground surface.

A good estimate of the reservoir properties as well as of the underlying and overlying low-permeable rocks is essentially needed to obtain realistic predictions. The model has also shown that the depth at which the rigid basement is set beneath the reservoir has a limited influence on the magnitude of the ground surface movement.

Finally it turns out that transversal anisotropy and larger values for the Poisson ratio of the clays may contribute to a significant increase of land subsidence.

Introduction

Among the various subsidence mechanisms which have recently been reviewed by Scott (1979), fluid withdrawal from subsurface resources plays probably the most important role. Areas where major land subsidence has occurred have been described by Poland and Davis (1969) and Poland (1972). A recent analysis by Chi and Reilinger (1982) focuses on the several locations where anomalous relative settlement has been recorded in the United States.

Although less common than water pumpage, gas/oil removal has nevertheless given a significant contribution to the list of case hystories of land sinking. Large settlements above gas/oil fields have been reported from Long Beach-Wilmington (California, U.S.A.), Goose Creek (Texas, U.S.A.),Groningen (The Netherlands), the river Po Delta (Italy), Bolivar Coast (Venezuela) and Niigata (Japan). A recent case with important social and economical implications is Ravenna (Italy) where the simultaneous withdrawal of groundwater from the upper aquifers and of gas from a number of deeper reservoirs scattered offshore and inland has created a precarious and complex situation.

Gas/oil production in subsiding areas is almost exclusively obtained from geologically recent sediments (post Eocene) at a depth which seldom exceeds 2000 m. Sediments usually consist of sand and clay. The sinking surface takes the shape of a bowl centered above the reservoir and the settlement quickly decreases with the distance from the production wells.

In complex geologic environment land subsidence due to fluid removal can be effectively analysed and predicted by the aid of mathematical models. However developing reliable prediction tools is still a challenge for the researchers working in this field. Due to the chronic shortage of essential data usually some basic assumptions are needed before a model can be realistically applied. In the model used to simulate the subsidence over the gas field of Groningen (Geertsma, 1973) a homogeneous and isotropic semi-infinite porous medium was assumed. This was warranted by the substantial lack of data relative to the soils overlying the gas bearing strata and by the large financial costs that are involved in providing such data. On the other hand layered sedimentary structures are quite common in the Quaternary depositional environment and an "a priori" analysis of the influence of the soil properties and their relative importance in controlling the subsidence rate is very useful in order to define correct investigation programs in subsiding areas above deep gas/oil reservoirs.

In the present paper the influence on land sinking of heterogeneity and transversal anisotropy of the soils which overly a gas/oil field located in a layered sedimentary structure is investigated with an elastic linear finite element model. An idealized disk-shaped reservoir sited at a depth between 1500 and 2000 m with an outer radius R = 2500 m and a thickness h = 65 m has been assumed. Fluid pressure decline has been taken equal to the reference value of 1 Kg/cm^2 evenly spread within the reservoir. In the overlying and underlying low permeable clays the water pressure decline is assumed to vary linearly with thickness from 1 to 0. The model has been run to provide the land subsidence on a radial profile for various structural sceneries where typical compressibilities for sand and clay progressively decrease with depth. The values used by the model fall within the range measured for the sands and clays of the river Po plain. The influence of transversal anisotropy, Poisson's ratio and location of the rigid basement has been explored as well.

After a short theoretical description of the finite element model, the paper gives some representative results showing the interrelation between land subsidence and soil structure.

Finite Element Model for Layered Sedimentary Structures

The finite element model developed to analyse the land subsidence in layered sedimentary structure belongs to the class of models originally pioniered by Lee and Shen (1969) and Sandhu and Wilson (1970). In their subsidence analysis over the gas field of Groningen, Geertsma and Van Opstal (1973) noted that "the finite element approach appeared too sophisticated a tool in relation to our very limited knowledge of the deformation properties of the sedimentary structure surrounding the reservoir". Thus they made use of the "tension center" approach which assumes a homogeneous and isotropic semi-infinite porous medium. However they did not consider that a finite element model is a very effective tool to investigate the relative importance of the information to which land sinking is theoretically related and especially so for the relationship between land subsidence and mechanical properties of the soil structure.

The present model is based on the following assumptions:

1) as a first approximation soils behave elastically

2) individual grains are incompressible

3) Terzaghi's principle holds. In other words all skeleton deformation is exclusively due to variations of the effective intergranular stress.

A pore pressure decline equal to 1 kg/cm^2 occurs in a disk-shaped reservoir characterised by:

depth of burial	$c = 1750$ m
thickness	$h = 65$ m
outer radius	$R = 2500$ m

In the overlying and underlying clays, whose thicknesses are equal to 60 m and 25 m respectively, the fluid pressure varies linearly from 1 to zero. No lateral aquifer is assumed. The rigid basement is set to a depth which is investigated by the model. On the outer boundary 20 km far from the reservoir axis zero vertical and horizontal displacements are prescribed.

If $p(x,y,z)$ is the final incremental pore pressure distribution within the sedimentary structure (schematically shown in Figure 1) and u_x, u_y and

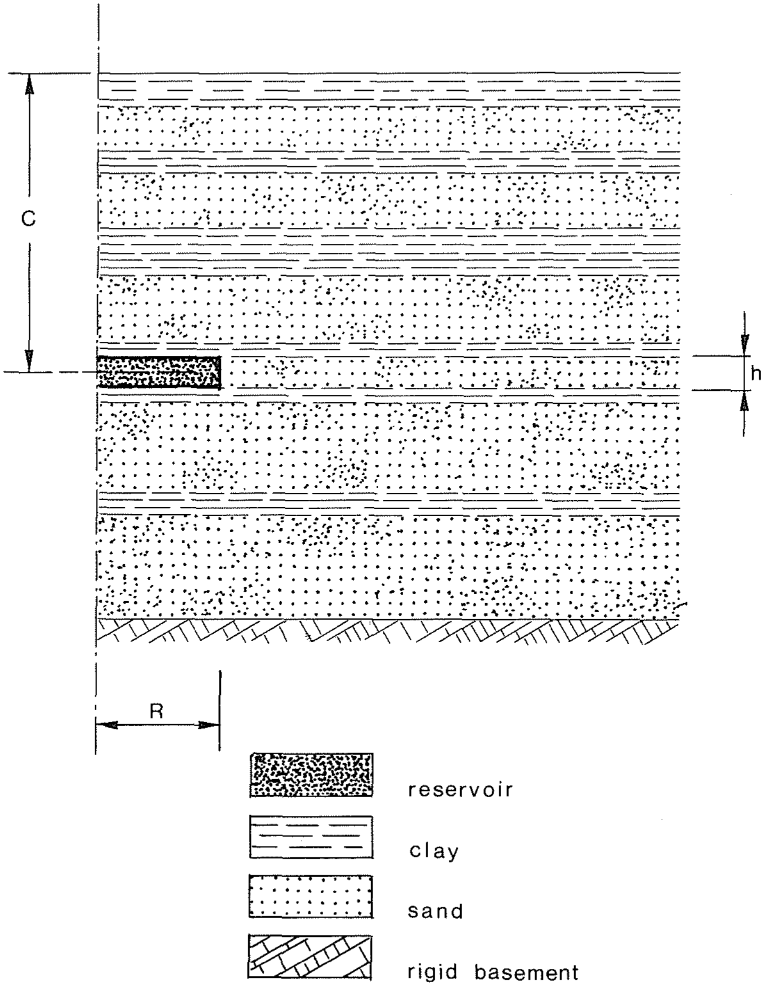

Figure 1: Schematic sedimentary structure of alternating clays and sands. The reservoir is assumed to be overlain and underlain by clays.

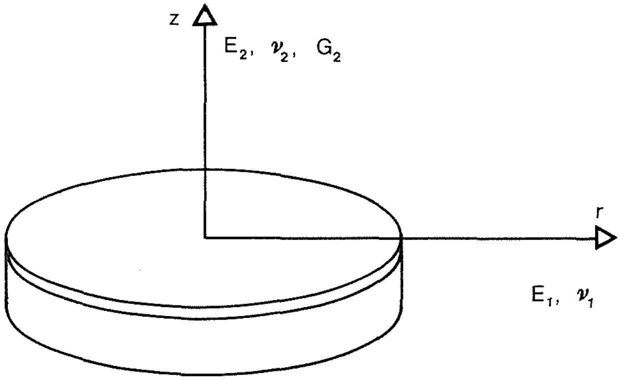

Figure 2: Scheme of a transversally anisotropic soil whose mechanic behavior is characterized by 5 distinct elastic constants

u_z are the components of the position vector measured from the initial reference state, the equilibrium equations for mechanically isotropic systems in terms of displacements may be written as (Gambolati , 1974):

$$\begin{aligned} G\nabla^2 u_x + (\lambda+G)\frac{\partial \varepsilon}{\partial x} &= \frac{\partial p}{\partial x} \\ G\nabla^2 u_y + (\lambda+G)\frac{\partial \varepsilon}{\partial y} &= \frac{\partial p}{\partial y} \\ G\nabla^2 u_z + (\lambda+G)\frac{\partial \varepsilon}{\partial z} &= \frac{\partial p}{\partial z} \end{aligned} \tag{1}$$

where $G = \frac{E}{2(1+\nu)}$ is the bulk shear or rigidity modulus, E is the Young modulus, ν is the Poisson ratio and λ is the Lamè constant equal to:

$$\lambda = \frac{\nu E}{(1-2\nu)(1+\nu)}$$

ε is the volume strain related to the linear strains by:

$$\varepsilon = \frac{\partial u_x}{\partial x} + \frac{\partial u_y}{\partial y} + \frac{\partial u_z}{\partial z}$$

and ∇^2 is the Laplace operator. In addition to the condition of zero total diplacement on the lower and outer boundaries, we prescribed the condition of zero normal and tangential stress on the ground surface (upper model boundary).

Eqs.(1) are solved by finite elements in the presence of radial symmetry allowing for transversal mechanical anisotropy. Hence each layer of soil is characterized by 5 elastic constants (Figure 2):

$$E_1, \nu_1, E_2, \nu_2, G_2$$

G_1 is dependent on E_1 and ν_1. The vertical soil compressibility α is defined as:

$$\alpha = \frac{1}{E_2}\left(1 - \frac{2\nu_2^2}{1-\nu_1}\,\frac{E_1}{E_2}\right) \qquad (2)$$

For isotropic soils ($E_1 = E_2 = E$; $\nu_1 = \nu_2 = \nu$) eq.(2) turns into the well known relationship (3):

$$\alpha = \frac{1}{E}\,\frac{(1+\nu)(1-2\nu)}{1-\nu} \qquad (3)$$

The porous medium is discretized into annular elements of triangular cross section. Elements are smaller inside and close to the reservoir where stress gradients are more pronounced (as a vertical cross section of a typical mesh see Figure 3). The model can treat up to 2500 nodal points. The model solver is based on the very accurate and efficient modified conjugate gradients (Gambolati, 1980). The model is currently run via telephone line on the IBM computer 3033 located at the CNUCE (Centro Nazionale Universitario per il Calcolo Automatico) of Pisa.

Results

Figure 4 and Figure 5 give the behavior of the vertical compressibility of soil α vs depth for sands and clays, respectively. The profiles are derived from data of Quaternary deposits of the eastern river Po plain published in earlier works by Brighenti (1964), Gambolati et al. (1974) and Brighenti and Fabbri (1982). Note that at the ground surface α_{sands} is two order of magnitude less than α_{clays} while at the depth of burial it is only three times smaller. Unless otherwise specified $\nu = 0.25$ is assumed everywhere in the layered structure. Correspondingly eq. (3) provides the E-values for isotropic soils to be fed into the model.

The land settlement η vs the dimensionless radius r/R for an entirely homogeneous medium is given by profile a) of Figure 6. The compressibility value is taken to be the same as that of the reservoir sands. If α decreases with depth as shown by Figure 4 (sandy structure) we obtain profile b), i.e. land sinking slightly increases. If the reservoir is overlain and underlain by clays the subsidence becomes significantly larger as may be seen

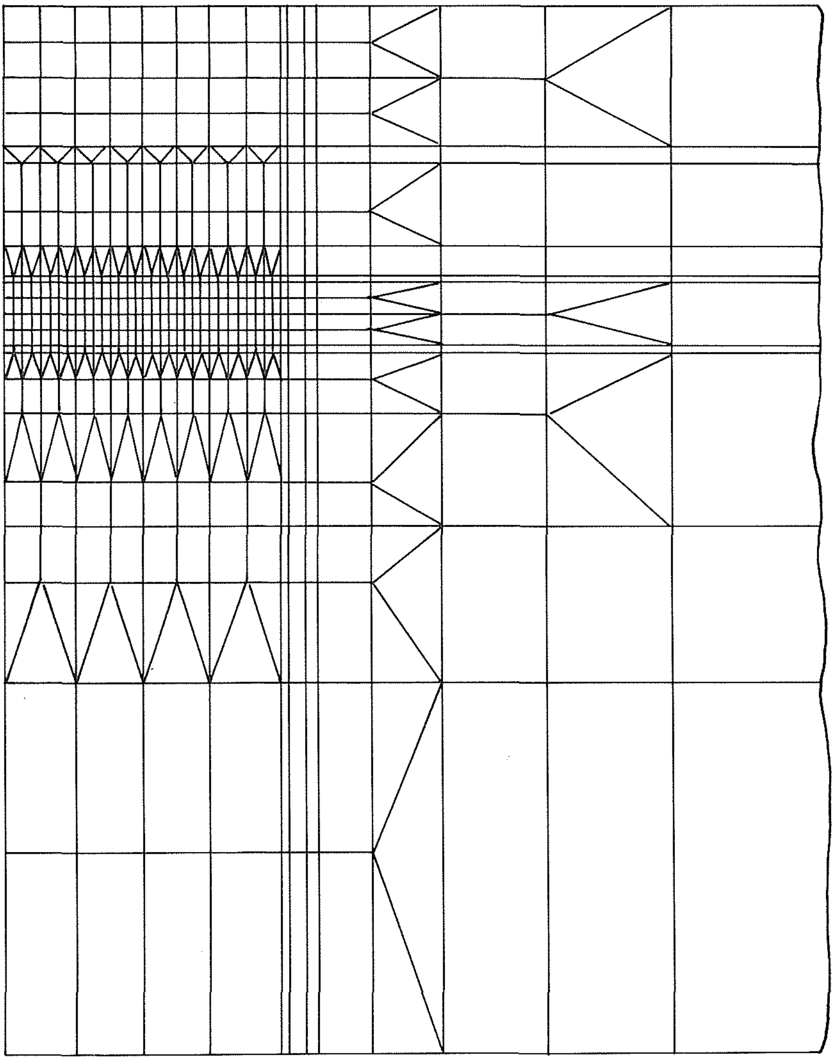

Figure 3: Triangular finite elements mesh to simulate land subsidence due to gas/oil removal from a disk-shaped reservoir in a sedimentary layered structure.
Meshes are generated automatically by the computer.

from profile c). Hence Figure 6 shows that a substantial contribution to land subsidence may be given by the compaction of the sediments surrounding the gas/oil bearing strata if the former are more compressible than the latter (in Figure 6 α_{clay} is three times higher than $\alpha_{reservoir}$ according to Figures 4 and 5).

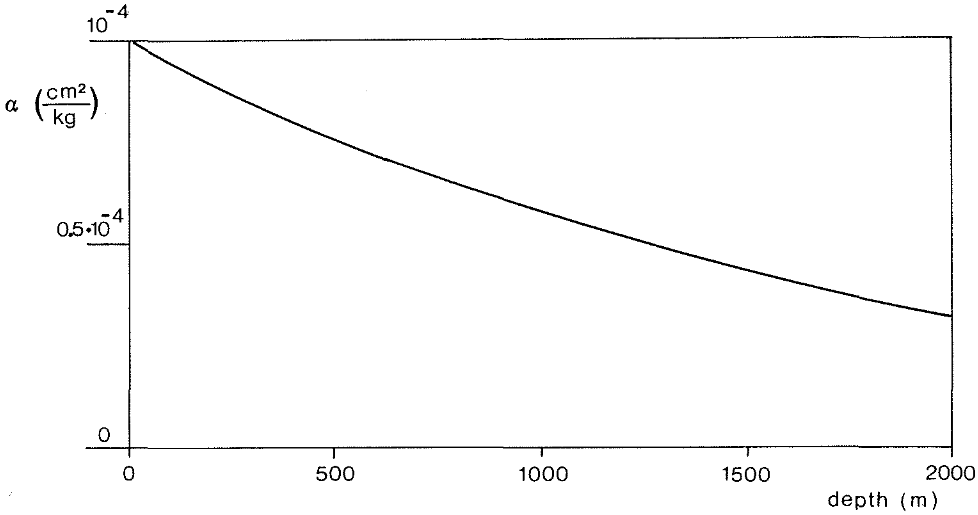

Figure 4: Behavior of vertical compressibility for sands vs depth

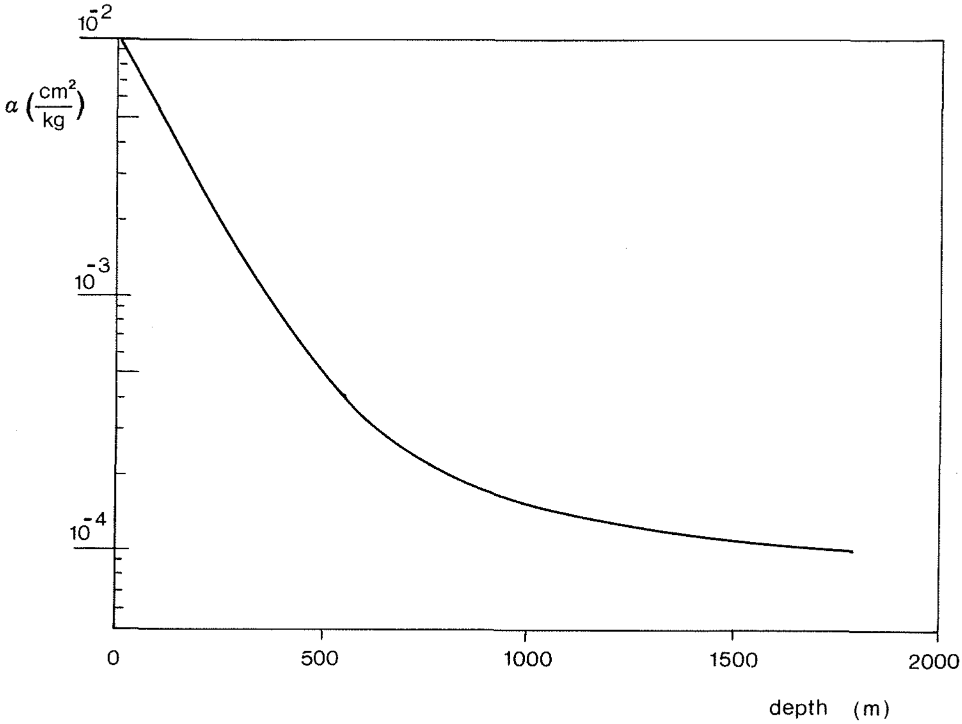

Figure 5: Behavior of vertical compressibility for clays vs depth

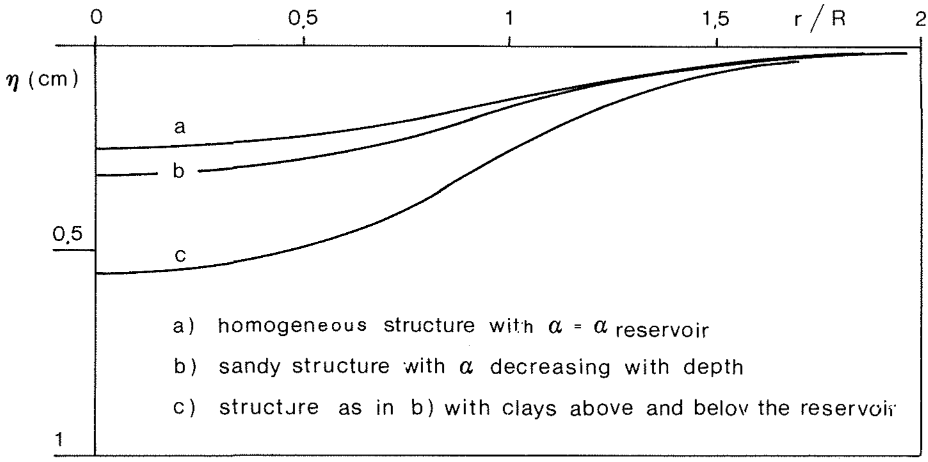

Figure 6: Land subsidence in homogeneous soils (a), sandy soils (b) and sandy soils with underlying and overlying clays (c)

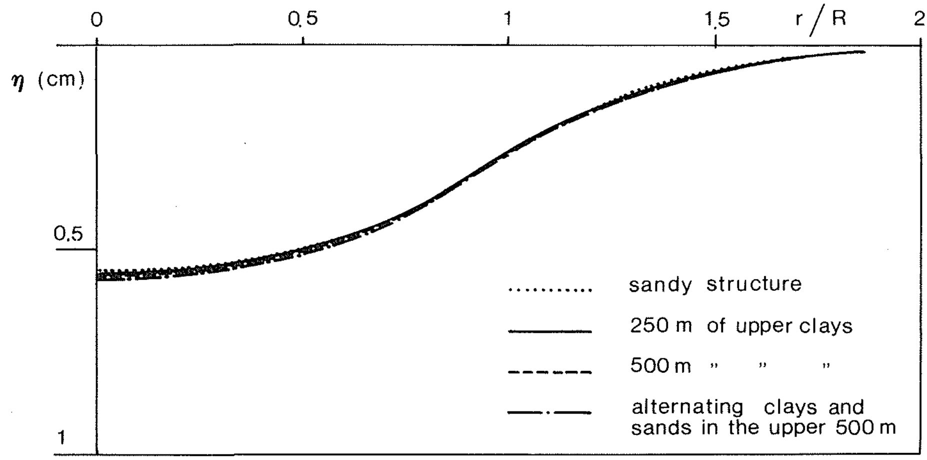

Figure 7: Influence on land subsidence of the stratigraphic composition of the upper 500 m

Profile c) is redrafted in Figure 7 and compared with the results obtained by setting:

- all clay in the upper 250 m
- all clay in the upper 500 m
- alternating clays and sands in the upper 500 m

It is worth noting that the actual nature of the upper sediments (either sand or clay) and their mutual position in the vertical stratigraphic profile has no practical influence on land settlement which is therefore primarily related, for a given reservoir geometry, depth of burial and fluid pressure decline, to the elastic constants of the reservoir and of the underlying/overlying clays. It turns out that for a realistic prediction of subsidence above deep gas/oil field good estimates of the reservoir mechanical poperties as well as of the top and bottom clays are much more important than those of the overlying sediments of the structure.

Figure 8 shows the influence on η of the depth of the rigid basement. The α distribution is the same as that of curve c) in Figure 6. The rigid basement is set beneath the clays underlying the reservoir, at 2500 m and at 20.000 m. The results indicate that η is not very sensitive to the position of the basement.

Land subsidence increases if the soils are transversally anisotropic as is shown in Figure 9 where the dashed profile is again profile c) of Figure 6. Setting $E_2 / E_1 = 2$ and $E_2/E_1 = 4$ (see Figure 2) leads to the dashed-dotted and to the solid curve, respectively. The assumption has been made that $\nu_1 = \nu_2$ and $G_2 = \frac{1}{2} \frac{E_2}{1+\nu_2}$

Finally the influence on η of Poisson's ratio is provided by Figure 10. Again with reference to profile c) of Figure 6 (the dashed profile here) it may be noted that increasing ν yields a decrease of η. In particular the dotted profile is obtained with $\nu = 0,25$ everywhere except for the overlying/underlying clays where ν has been set equal to 0.45. A larger value for Poisson's ratio in the low permeable sediments adjacent to the reservoir leads to a reduction of land subsidence.

Conclusion

The analysis by the finite element model has been carried out primarily to explore the influence of the deformation parameters of the sedimentary layered structure on land subsidence caused by gas/oil production from a deep circular reservoir. For a given geometry, depth of burial and fluid pressure decline the results from the model stress the following conclusive remarks:

a) Land subsidence is basically related to the compressibility of the gas/oil bearing strata and of the adjacent overlying/underlying clays.

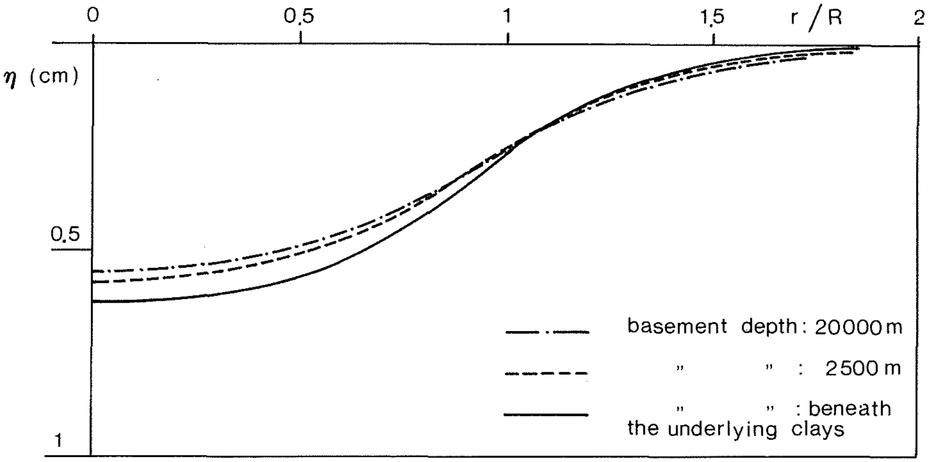

Figure 8: Influence on land subsidence of the position of the rigid basement

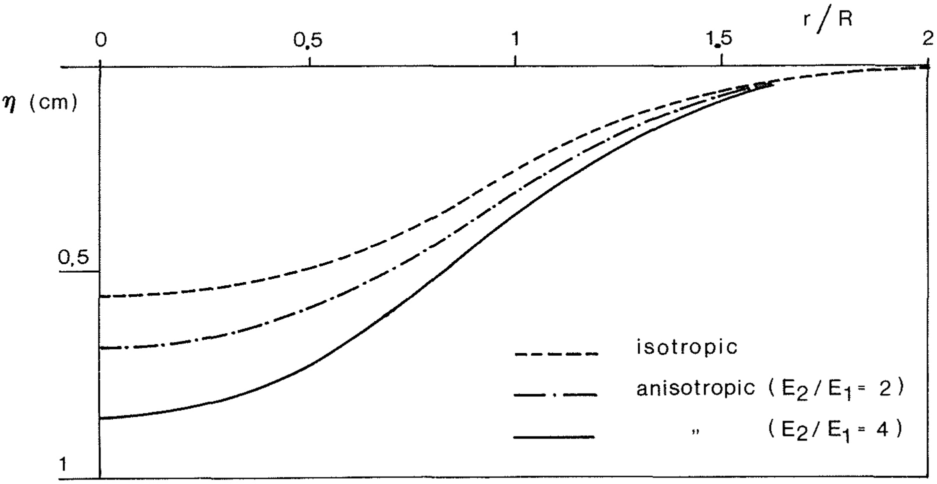

Figure 9: Influence on land subsidence of transversal mechanical anisotropy

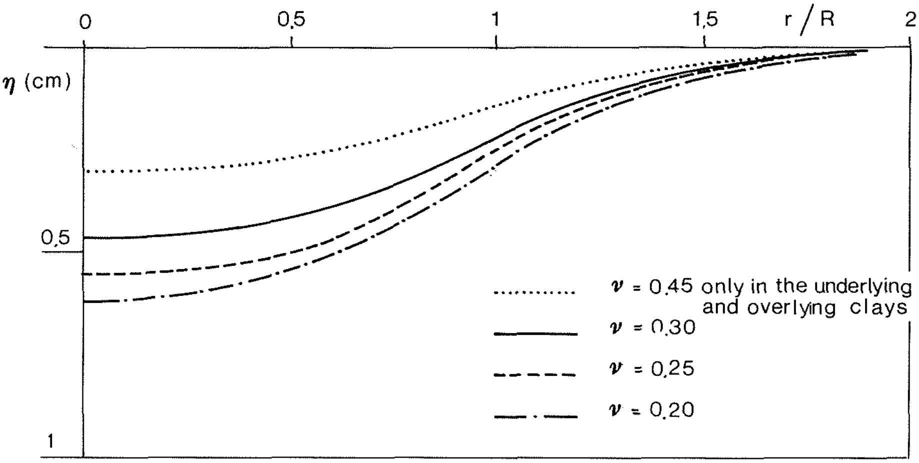

Figure 10: Influence on land subsidence of Poisson's ratio

b) An accurate description of the upper soils with a good determination of their mechanical properties is not needed since land settlement is rather insensitive to the deformation properties of the sediments where no pore pressure decline occurs.

c) The position of the rigid basement (just below the underlying clays or at a larger depth) has a small influence on land sinking.

d) Possible transversal anisotropy yields larger subsidence values.

e) Increasing Poisson's ratio (e.g. in the overlying and underlying clays) leads to a decrease of the subsidence profile.

On summary it appears that any program intended to provide realistic predictions of land subsidence caused by gas/oil production with a mathematical model has to focus primarily on the reservoir and the overlying/underlying clays and to give representative records of their mechanical behavior.

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ANALYSIS OF LAND SUBSIDENCE USING AN ELASTOPLASTIC CRITICAL STATE SOIL MODEL

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Abstract

A numerical simulation of a simple idealized subsidence problem is performed. Special reference is made to the irreversible deformations occurring both in reservoir areas and overburden layers during the fluid extraction process. A non-associative elastoplastic constitutive law for soil behaviour, based on the principles of Critical State Soil Mechanics, is described. A fully coupled finite element technique employing the soil model suggested is used in the analysis. The topics discussed include the appearance of plastic zones, influence of prestress, formation of soil discontinuities and the effects of depth and compressibility of the reservoir on the subsidence levels. The difficulties related to modelling soil behaviour under the effects of reverse loading, high temperature changes and long term deformation, are briefly mentioned.

Introduction

Surface subsidence due to fluid withdrawal is related to reservoir compaction via the mechanical response of the overburden. The highly plastic nature of most soils suggests that, in order to model deformations during compaction processes, it is necessary to consider the elastoplastic behaviour of geologic material. Compaction-subsidence studies have, in the past, neglected the importance of large irreversible deformations and the effect of the stress history. In some cases, where elastoplastic laws have been used, the soil parameters are difficult to evaluate and this limits the applicability of such models.

Critical State Soil Mechanics has provided the basis for building elastoplastic stress-strain relationships which explain different features of soil behaviour with a reduced number of parameters. The equations are based on the experimental observation of soil response and the model parameters can all be determined using standard soil testing procedures. This represents an advantage over the usual problem of providing input data for complex behavioural models. The critical state laws give good quantitative predictions for the behaviour of a large range of soil materials and can be considered valid for general loading paths.

In this study, a critical state constitutive law is coupled with the flow equation to study deformation mechanisms related to compaction and subsidence.

Critical State Constitutive Laws

Critical State Soil Mechanics can be considered as a unifying criterion for understanding the complex mechanical behaviour of soil. In the light of critical state ideas, the state of a soil is determined by its specific volume (or state of volumetric strains) and the effective stresses acting on it. As shearing develops, and if the restraining conditions permit, the density of soil changes and tends to a constant value called the critical density which depends only on the effective pressure. Once this density is achieved, the soil is said to have reached the critical state and continues to deform with constant strength and at constant volume. Details of the critical state theory have been published by Schofield and Wroth (1968) and Silva-Pérez (1984) presents the concepts in the context of subsidence analysis. Therefore only an outline of the concepts necessary to understand the constitutive relations will be given here.

Experimental evidence shows that for a wide range of soils there is a unique curve of critical states in (P', q, $\varepsilon_v{}^P$) space. Where P', q and $\varepsilon_v{}^P$ are defined as follows:

$$P' = (\sigma'_{11} + \sigma'_{22} + \sigma'_{33})/3 \qquad \text{(isotropic effective stress)}$$

$$q = \left[(3/2)\, S'_{ij}\, S'_{ji}\right]^{1/2} \qquad \text{(deviatoric stress)}$$

$$S'_{ij} = \sigma'_{ij} - P'\, \delta_{ij}$$

$$\varepsilon_v{}^p = \varepsilon_{11}^p + \varepsilon_{22}^p + \varepsilon_{33}^p \qquad \text{(plastic volumetric strain)}$$

The following quantities will also be used:

$$\bar{\varepsilon} = \left[(2/3)\, e_{ij}\, e_{ji}\right]^{1/2} \qquad \text{(deviatoric strain)}$$

$$e_{ij} = \varepsilon_{ij} - (1/3)\, \varepsilon_v\, \delta_{ij}$$

with

ε_{ij} = total strain

ε^e_{ij} = elastic strain

ε^p_{ij} = plastic strain

The critical states can be represented by two simple empirical relations (Fig. 1):

$$q = MP'_c \tag{1}$$

$$P'_c = P'_{co} \exp\,(\beta\, \varepsilon^p_v) \tag{2}$$

where M, P'_{co} and ß are the critical state parameters.

It has been stated that when soils are sheared they tend to a critical state. Following this assumption, the curve of critical states shown in Fig. 1b can be considered as the boundary between two types of behaviour. Soils that are initially dense, to the left of the curve of Fig. 1b, have a strong tendency to dilate or to generate negative pore pressures if drainage is not allowed. Soils that are initially loose, to

FIG. 1a

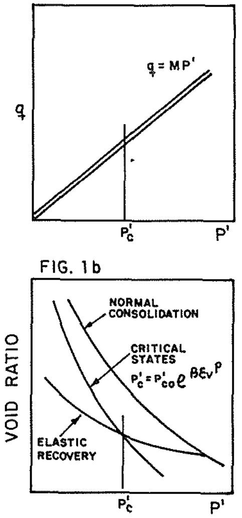

FIG. 1 - CRITICAL STATES.

the right of the curve of critical states, tend to contract when sheared or to generate transient positive pore pressures. Atkinson and Bransby (1978) provide a useful collection of experimental data showing the position of the critical states for both sands and clays.

The existence of a unique ultimate or critical state, depending only on the effective pressure, can be used to formulate an equation of energy dissipation which, together with the concepts of elasticity and plasticity, can be extended to provide constitutive laws for soil behaviour. The energy, E, dissipated by a deforming soil can be written as:

$$E = \sigma'_{ij} \, \dot{\varepsilon}^{p}_{ij} = P' \, \dot{\varepsilon}_{v}^{p} + s'_{ij} \, \dot{e}^{p}_{ij} \tag{3}$$

Assuming that the strain increments are colinear with the stresses, Eq. (3) becomes:

$$E = P' \, \dot{\varepsilon}_{v}^{p} + q \, \dot{\bar{\varepsilon}}^{p} \tag{4}$$

The dissipated energy at the critical state is:

$$E = MP' \, \dot{\bar{\varepsilon}}^{p} \tag{5}$$

Roscoe et al (1963) present experimental evidence showing that Eq. (5) may be assumed valid for any stage of deformation, so that the complete equation for the dissipation of energy is:

$$P' \, \dot{\varepsilon}_{v}^{p} + q \, \dot{\bar{\varepsilon}}^{p} = MP' \, \dot{\bar{\varepsilon}}^{p} \tag{6}$$

The plastic potential, g_1, can be obtained by direct integration of Eq. (6), giving:

$$g_1(P', q) = q/(MP') + \ln P' \tag{7}$$

with:

$$\dot{\varepsilon}_{v}^{p} = \lambda \, (\partial g_1/\partial P') = \lambda/MP' \, (M - q/p') \tag{8}$$

$$\dot{\bar{\varepsilon}} = \lambda \, (\partial g_1/\partial q) = \lambda/MP' \tag{9}$$

and in terms of individual components:

$$\varepsilon_{ij} = \lambda/MP' \left[(3/2) \, s'_{ij}/q + (1/3) \, \delta_{ij} \, (M - q/P')\right] = (\lambda/MP')H_{ij} \tag{10}$$

In order to complete the mathematical description of the plastic behaviour of soil material, it is necessary to determine the level of the stresses needed to cause yielding. Early critical state models were built on the basis of a stability hypothesis suggested by Drucker (1964). This leads to an associative type of stress-strain law (plastic potential identical to the yield function). These models give fair quantitative predictions for loose or normally consolidated clayey soils. However, as the overconsolidation ratio becomes higher the predictions worsen. The models underestimate deformations at low shear forces and predict exceedingly large values of the "peak resistance". Also, problems were encountered for stress paths near the P' axis due to energy dissipation changes for isotropic consolidation processes. In an attempt to overcome these difficulties, modified versions

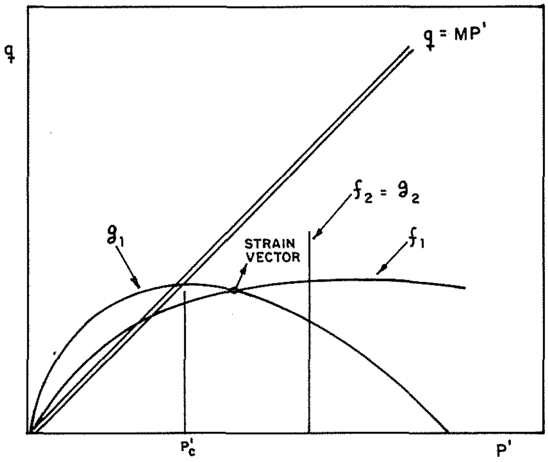

FIG. 2 - YIELD FUNCTIONS AND PLASTIC POTENTIALS FOR THE CRITICAL STATE MODEL.

of the original equations have been suggested (Burland, 1967; Roscoe and Burland, 1968).

The discrepancy between the observed behaviour of soil and the early critical state models, appeared to be due to the hypothesis of normality of the strain vector to the yield curve (arising from the stability criterion). Granular materials show some dependency on the stress path followed and behave more like non-associative behavioural models. For this reason Hujeux (1979) rejected the hypothesis of an associative law and, keeping the plastic potential of Eq. (7), suggested the following yield function:

$$f_1\ (P',\ q,\ \varepsilon_v{}^p,\ \bar{\varepsilon}\ ^p) = q - MP'\ \left[1 - b\ \ln(P'/P_c^{'})\right]\ \bar{\varepsilon}\ ^p/(a + \bar{\varepsilon}\ ^p) \qquad (11)$$

where a is related to the initial shear modulus of the soil and b is introduced to regulate the "peak" resistances.

To account for energy changes near the isotropic axis, a second associated yield function is assumed, whose expression is:

$$g_2\ (P',\ \varepsilon_v{}^p) = f_2\ (P',\ \varepsilon_v{}^p) = P' - \alpha\ P_c^{'} \qquad (12)$$

where α is an empirical parameter. The yield curves and plastic potentials are represented in Fig. 2.

The elastic part of the deformation is given by:

$$dP' = K_oP'\ \varepsilon_v^e \qquad (13)$$

$$dq = 3\ G_oP'\ \bar{\varepsilon}\ ^e \qquad (14)$$

where K_oP' is the bulk modulus and G_oP' the shear modulus.

The elastoplastic constitutive law, conceptually described above, has shown satisfactory agreement with experimental observation for a wide range of soils and testing procedures. Figs. 3 and 4 show applications of the model to the behaviour of a frictional and a cohesive soil respectively. It

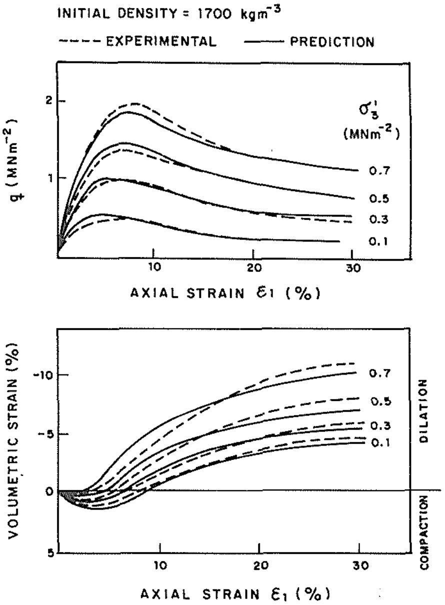

FIG. 3 - DRAINED TRIAXIAL BEHAVIOUR OF THE HOSTUN SAND (EXPERIMENTAL DATA AFTER AL-ISSA, 1973)

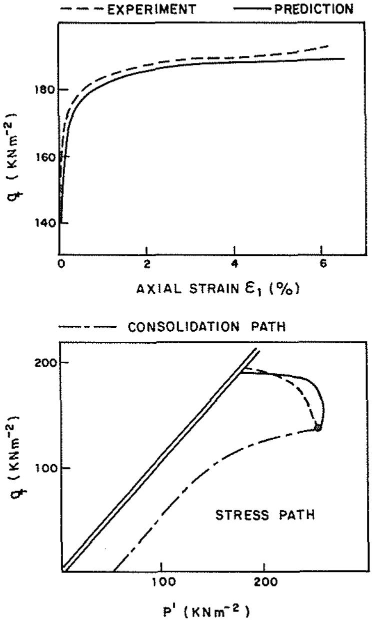

FIG. 4 - UNDRAINED TRIAXIAL BEHAVIOUR OF NORMALLY CONSOLIDATED ORINOCO CLAY.

can be observed that, considering the different characteristics of both soils, the predictions are good.

In what follows, a fully-coupled finite element technique is used to calculate the surface subsidence over a model reservoir. The critical state constitutive relations are used in the analysis in order to include elasto-plastic effects in the reservoir and overburden layers.

Application to a Subsidence Problem

Significant withdrawal of fluid from underground reservoirs usually gives rise to superficial subsidence. The mechanical behaviour of the system is controlled by consolidation of the reservoir and deformation of the upper layers of soil. Compaction of the reservoir is induced by an increase in the effective stresses due to the reduction in pore pressure caused by pumping. The overlying mass of soil tends to follow the movement of the reservoir, leading to the generation of additional shear stresses outside the production zone. Critical state theories bring together the consolidation and shearing behaviour of soils, and for this reason they constitute a convenient framework to study the deformation mechanisms caused by production. A finite element solution algorithm, using Darcy's law and the previously suggested rheological model for the soil skeleton, has been chosen. The fully coupled finite element technique used for the present analysis has been described in detail elsewhere (Hujeux, 1979), therefore only a summary of the numerical results obtained will be given here.

In the finite element simulation performed, a plane of symmetry representing a section of the Earth's crust surrounding an axisymmetric well was isolated and represented by a mesh as shown in Fig. 5. The elements consist of 8-node quadrilaterals and 6-node triangles with 2 x 2 and 3 point integration respectively. The quality of the solution depends on how the

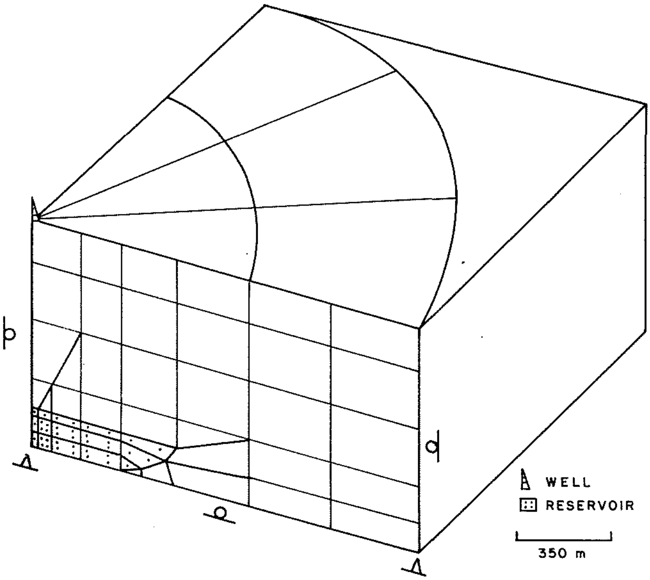

FIG. 5 - SUBSIDENCE MODEL CONFIGURATION.

discretization is performed and because a simple mesh is employed, results must be interpreted qualitatively and in the light of general mechanisms of deformation.

The input data includes the material properties and the effective pressure reduction values, below hydrostatic, in those elements which correspond to the reservoir area. The complete set of input parameters used is given in Table 1. The soil is assumed to be saturated with incompressible fluid

	Reservoir	Soil Stratum 1 $o < Z < 650m$	Soil Stratum 2 $650 < Z < 775m$
Young's modulus (MN/m^2)	250.0	640.0	640.0
Poisson's ratio	0.316	0.316	0.316
Horizontal permeability (m/month)	0.287	1.0 x 10^{-7}	1.0 x 10^{-7}
Vertical permeability (m/month)	0.068	1.0 x 10^{-7}	1.0 x 10^{-7}
Coeff. of earth pressure at rest	0.7	0.5	0.7
Specific weight of fluid (MN/m^3)	0.00981	0.00981	0.00981
Saturated unit weight of soil (MN/m^3)	0.0118	0.0118	0.0118
α	1.5	1.5	1.5
M	1.2	1.557	1.557
β	29.0	35.0	35.0
P_{co} (MN/m^2)	0.319	0.596	0.596
a	0.0075	0.0075	0.0075
b	1.0	1.0	1.0
Effective pressure reduction in reservoir (MPa)	2.0		

TABLE 1 - SYSTEM PARAMETERS

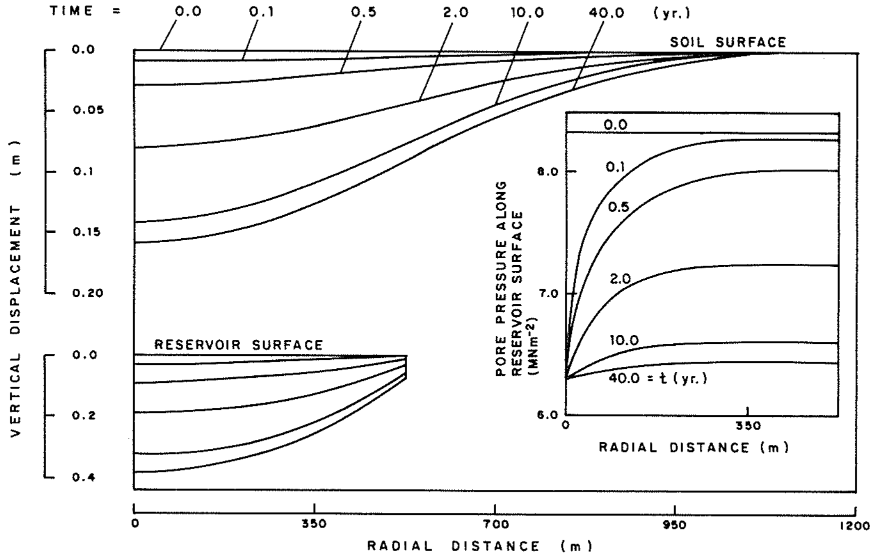

FIG. 6 - EVOLUTION OF SETTLEMENT AND PRESSURES.

and with the water table at ground level. The pumping effect is achieved by suddenly increasing the effective pressure at the centre of the well.

Fig. 6 shows the settlement of the reservoir surface and the shape of the subsidence bowl after different periods of time, together with the corresponding drops in pore pressure along the reservoir surface. Fig. 7 indicates the zone of influence of deformations and the areas which, given the system dimensions and soil properties, have experienced plastic flow. The transition from low to high ground represents a zone where high shear stresses develop, causing the yield curve to be reached. For a specific site the limit of elastic deformation will be determined by the amount of prestress that the soil has experienced before the start of production. Surface

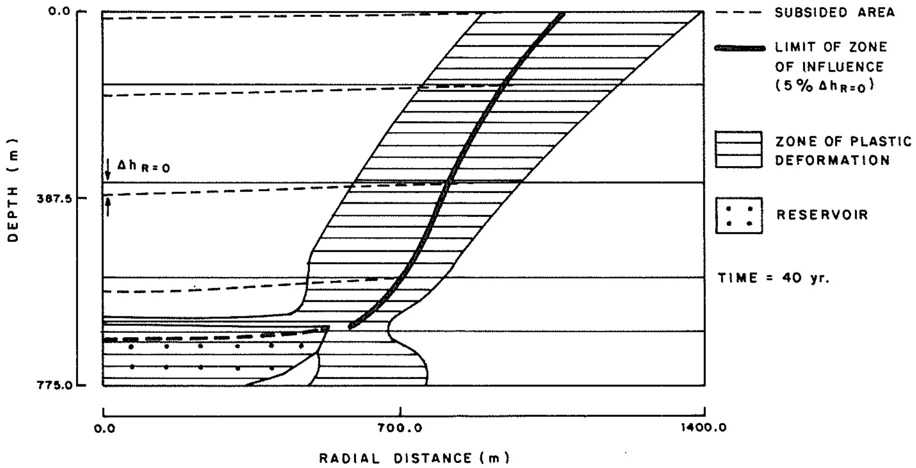

FIG. 7 - ZONE OF INFLUENCE

erosion, water table fluctuations or earthquake shaking may cause the initial stresses to be considerably lower than yield. It has also been suggested (Kosloff et al, 1980) that the role of additional compressive or tensile lateral stresses, of tectonic origin, will have an important effect on the size and shape of the subsidence bowl. These stresses may be affected by local flexure of the soil layers, extensive faulting or sloping ground.

Bending of the overburden layers will cause a reduction of lateral stresses in high inflexion areas near the limit of the zone of influence. Fig. 8 shows the changes of lateral stress, as compared to the "at rest" values, at three different depths for the last stage of calculation (40 years). This effect may produce tensile stresses which may give rise to soil discontinuities and surface cracking. The appearance of cracks, at the edge of the bowl of subsidence, is common to many underground mining and fluid extraction areas and is one of the most damaging environmental effects of compaction phenomena. Cracks are not only limited to dry ground but have been recently observed in areas where production is performed in shallow waters. The depth of the soil discontinuities is difficult to estimate with reasonable accuracy. However, there is evidence from direct measurements that is some areas they extend to more than 50m depth. In addition, the presence of high pressure gas near the cracking zones may indicate that in oil production areas the discontinuities can extend considerably deeper.

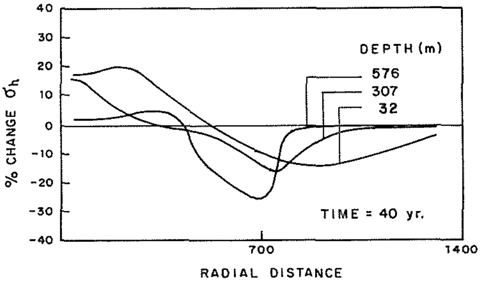

FIG. 8 - CHANGE IN LATERAL STRESSES

The effect of reservoir compressibility on the fluid recovery and subsidence levels is shown in Fig. 9. The fluid withdrawals are caused by the same drop in pressure and the cumulative production is presented as a function of the isotropic compaction coefficient ß. The results obtained indicate the effect of compaction as a drive mechanism. This is particularly important in zones where exploitation of heavy oils from uncemented sands is performed. In these areas a high compaction coefficient will increase the effectiveness of recovery and improve the economics of production. This contribution will vary with the geometry and the mechanical properties of the reservoir and its fluids.

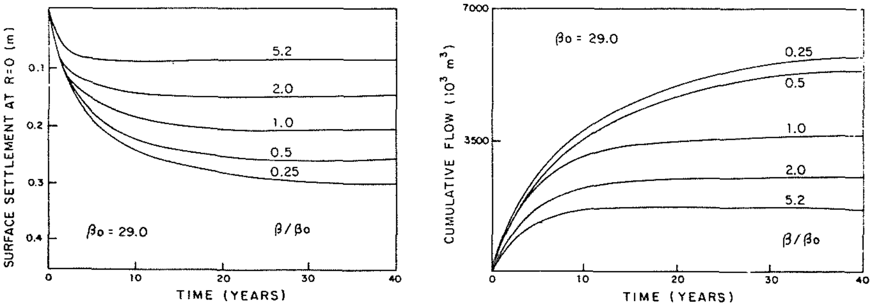

FIG. 9 - EFFECT OF RESERVOIR COMPRESSIBILITY.

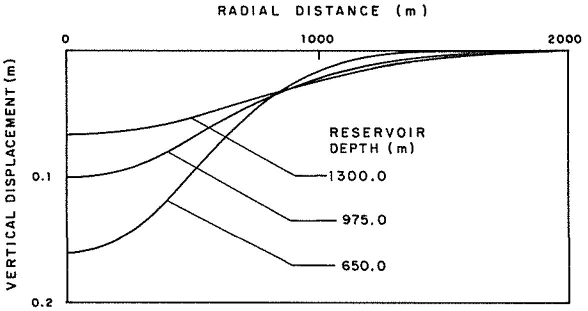

FIG. 10 - EFFECT OF DEPTH OF BURIAL ON SUBSIDENCE LEVELS.

The surface subsidence for similar reservoirs, showing the same amount of compaction, but located at different depths was also simulated and the results are presented in Fig. 10. As expected, subsidence increases as the depth of burial decreases. This supports the evidence, obtained from previous subsidence simulations (Geertsma, 1973), that the ratio between surface subsidence and reservoir compaction is determined by the ratio between depth of burial and the lateral extent of the reservoir.

Additional Considerations

The critical state constitutive law described above is strictly applicable to the behaviour of relatively shallow reservoirs consisting mainly of sand and clay soils. For deeper reservoirs there may be considerable crushing and plastic deformation at the grain contacts. This activates additional energy dissipation mechanisms which may affect the predictions of the model suggested. An attempt was made by Gerogiannopoulos and Brown (1978) to extend the critical state theory to rock behaviour by including the effect of dissipation mechanisms different to friction. A similar approach could be used to model behaviour of soil at high earth pressures.

Until now, reference has only been made to monotonic loading due to continuous production. However, it is sometimes desirable to simulate the soil response under reverse loading. This condition may arise when continuous or cyclic repressurization of the reservoir is performed by the injection of fluids. In such cases it is necessary to consider the marked hysteretic behaviour of soils when subjected to reverse loading. The isotropic hardening which was previously adopted is not appropriate to model that type of response because it implies the existence of an exceedingly large elastic domain. The assumption of elastic unloading may lead to underestimation of the amount of rebound and of the extensive localized deformations which may occur near repressurization wells. To account for the anisotropic behaviour of soils subject to repeated loading, a possible generalization of the critical state models for cyclic behaviour was suggested by Hujeux and Aubry (1981).

Extraction of fluids from underground reservoirs causes soil deformations during periods of time which are considerably higher than times involved in conventional soil laboratory testing. For this reason, secondary compression may introduce a significant error in the estimates of final compaction and ultimate subsidence based on short term laboratory measurements. This effect must be investigated in order to determine the accuracy of extrapolating short-term creep experiments to the long-term behaviour of geologic material during compaction processes. High temperature fluctuations will also alter the rheology of soil by introducing changes in the chemical and mechanical properties of grains and fluids. A range of deformation mechanisms may be activated and this suggests that, for general compaction/subsidence analysis, the use of a set of constitutive laws based on a single energy dissipation mechanism is too simplistic.

Conclusions

The intention of this paper has been to perform a subsidence simulation of a simple idealized reservoir-overburden system. The attention has focused on the elastoplastic properties of the soil skeleton rather than on the details of fluid flow. The occurence of plastic deformation, both in the reservoir and overlying layers, has been discussed in terms of the geometry of the problem and the effect of historic and tectonic stresses. The role of prestress emphasizes the convenience of employing elastoplastic constitutive laws for subsidence modelling. It was also attempted to link, in qualitative terms, common phenomena related to compaction with the numerical results obtained. In this respect, the formation of surface cracks and soil discontinuities has been associated to the reduction of earth pressures at various depths near the limit of the zone of influence of deformations. The effect of reservoir compressibility on fluid production and depth of burial on subsidence levels has also been discussed. The tendencies shown by the calculations agree with field observations and previous subsidence simulations.

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EVALUATION OF A TECHNIQUE FOR SIMULATING A COMPACTING AQUIFER SYSTEM IN THE CENTRAL VALLEY OF CALIFORNIA, U.S.A.

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Abstract

Large volumes of water have been pumped from the Central Valley aquifer system since the early 1900's. Water levels in the most heavily pumped areas had declined as much as 120 m by 1970. These large water-level declines resulted in approximately 21,000 hm^3 of water released by inelastic compaction of numerous compressible fine-grained deposits.

The principal technique used to evaluate the aquifer system was a three-dimensional computer program that solves the basic ground-water flow equation. The program was modified to incorporate water released by inelastic compaction of the fine-grained deposits by making storage a function of hydraulic head. The computer-simulated volume of water released from inelastic compaction for the period from 1961 through 1977 was 6 percent of the estimated volume. The technique could be used in other areas where water is released as a result of inelastic compaction of fine-grained deposits.

Introduction

The Central Valley of California encompasses an area of nearly 50,000 km^2 and extends from near Red Bluff at the north end to the vicinity of Bakersfield at the south end, a distance of 650 km (fig. 1). The valley is commonly divided into two parts, each drained by a major river. The northern one-third is the Sacramento Valley and the southern two-thirds, the San Joaquin Valley. The Central Valley is a long northwestward-trending asymmetric structural trough that is filled with sediment. The fresh ground-water reservoir is generally within the continental deposits that consist predominately of lenses of gravel, sand, silt, and clay.

Numerous lenses of fine-grained deposits (silt, sandy silt, sandy clay, and clay) are scattered throughout the valley and, as determined from electric logs, constitute between 40 and 60 percent of the total thickness penetrated by wells. Although most of these lenses are not widespread, a few major lenses have been mapped in the valley, principally beneath the axis of the San Joaquin Valley. The fine-grained lenses restrict vertical flow within the aquifer system.

The Central Valley is ideal for farming because of its fertile soil and long growing season. The amount of water required to support farming activities averages about 27,000 hm^3/yr. The water is obtained from two sources: streams and ground water. Streams that enter the valley, predominantely from the east side and the north end (fig. 1), account for about one-half of the total water used; streamflow is diverted via canals to areas of farming. Ground water is used primarily where surface-water supplies are not available or are insufficient to support farming activities. About 80 percent of the total ground water pumped in the Central Valley is in the San Joaquin Valley, whereas only 20 percent is pumped in the Sacramento Valley.

The large demand for ground water has placed a considerable stress on the aquifer system, particularly in the San Joaquin Valley where surface water is less plentiful. Ground-water pumpage has exceeded recharge in several

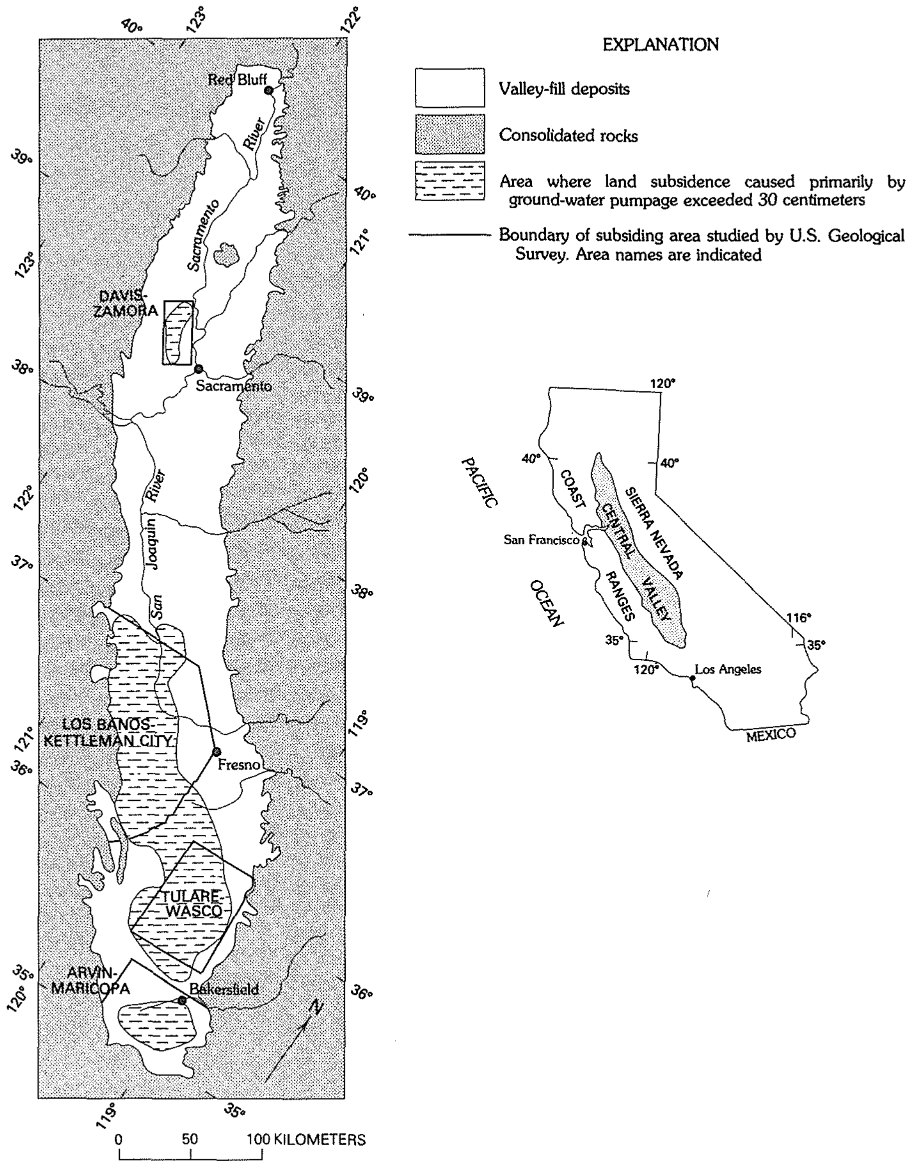


FIG. 1 Location of the Central Valley of California and areas of land subsidence due to ground-water pumpage.

parts of the valley and, between the early 1900's and the late 1960's, has caused water levels in a few areas to decline more than 120 m. These large water-level declines increased the effective stress (grain-to-grain load) in the aquifer system and, coupled with the numerous lenses of compressible fine-grained deposits, has caused the greatest volume of land subsidence in

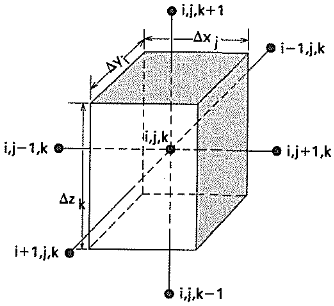


FIG. 2 Node array for finite-difference formulation.

the world due to ground-water pumpage (Joseph F. Poland, U.S. Geological Survey, oral commun., 1982). More than 13,500 km^2 have subsided at least 30 cm, and at one location, subsidence exceeded 8 m (Ireland and others, 1982). The estimated amount of water released from inelastic compaction is approximately equal to the volume of land subsidence, which from 1920 through 1977 was about 21,000 hm^3, of which about 9,000 hm^3 occurred from 1961 through 1977. Subsidence in the San Joaquin Valley has greatly decreased since about 1968 because of surface-water imports from canals and consequent reduction in pumping. Water levels in the Los Banos-Kettleman City area (fig. 1) recovered as much as 60 m from 1968 to 1975.

A regional analysis of the Central Valley aquifer system was done as part of the Regional Aquifer Systems Analysis, a National program of the U.S. Geological Survey. The purpose of this paper is to (1) discuss the technique that was used to incorporate the release of water from inelastic compaction of the fine-grained deposits into a computer model of ground-water flow for the aquifer system, (2) compare the simulated and estimated land subsidence during the period of analysis (from 1961 through 1977), and (3) discuss the differences in the relation between pumpage and inelastic compaction among major subsiding areas in the Central Valley.

Method of Analysis

The principal technique used to simulate ground-water flow in the Central Valley aquifer system was a three-dimensional finite-difference computer program written by Trescott (1975). The three-dimensional equation that the program solves can be written as follows:

$$S\ dh/dt + w(x,y,z,t) = d/dx(Kxx\ dh/dx) + d/dy(Kyy\ dh/dy) + d/dz(Kzz\ dh/dz)\ ,$$

where S = specific storage (L^{-1});
h = hydraulic head (L);
t = time (T);
w = volumetric flux of recharge or discharge per unit volume (L^{-1});
x,y,z = cartesian coordinates (L);
Kxx, Kyy = hydraulic conductivity in the principal horizontal directions (L/T); and
Kzz = hydraulic conductivity in the vertical direction (L/T).

The continuous derivatives in the flow equation are replaced with finite-difference approximations at a point or node. Surrounding each node is a block with dimensions x, y, and z in which the hydraulic properties are assumed to be uniform (fig. 2). The result is N number of unknown head values at the nodes and N number of equations, where N is the number of blocks that represent the aquifer system. The time derivative dh/dt is approximated by the backward difference procedure (Remson and others, 1971, p. 78). The approximation for each node is as follows:

$$(h_1 - h_0)/ \Delta t \ ,$$

where h_0 = the hydraulic head at the beginning of a time step (L);
h_1 = the hydraulic head at the end of a time step (L), which is unknown; and
Δt = the time-step interval (T).

The program solves the unknown head for each time step using a strongly implicit procedure (Trescott, 1975, p. 11); this is done by iterating through the finite-difference equations for each node until the head change between the previous iteration and the current iteration is less than a specified amount for all nodes. Once this criterion is met, the program advances to a new time step, and the process of computing heads at each node is repeated.

The program was modified to account for the release of water from the inelastic compaction of the fine-grained beds in the aquifer system with a technique similar to the one used by Meyer and Carr (1979) near Houston, Texas. The water is accounted for by changing the storage coefficient of each model block from an elastic value to an inelastic value. The elastic storage coefficient for each block is the sum of (1) the specific storage of the fine-grained deposits times their thickness and (2) the specific storage of the coarse-grained deposits times their thickness. The change from one storage value to another was set as a function of head. A head value (referred to as critical head) that coincides with the maximum effective stress to which the deposits had been previously subjected was assigned to each model block. If the initial hydraulic head (starting water level) is above the initial critical head, then the elastic storage coefficient is used until the hydraulic head falls below the critical head (fig. 3). When the computed hydraulic head declines below the critical head in a model block, the storage coefficient changes from an elastic value to an inelastic value for the fine-grained deposits at the beginning of the next time step. The inelastic storage coefficient is kept until the computed hydraulic head begins to recover, then the storage coefficient switches from inelastic back to elastic at the next time step and the hydraulic head at which recovery started is recorded as the new critical head. When the computed hydraulic head in a model block declines below the new critical head, the storage coefficient switches back to an inelastic value, again at the beginning of the next time step, and the cycle repeats itself (fig. 3). Land subsidence is calculated by multiplying the head decline below the critical head during each time step times the inelastic storage coefficient. Values of subsidence for each block were accumulated over the simulation period.

The program modification to account for release of water from inelastic compaction has a few drawbacks. First, the change in head in an aquifer system propagates slowly through the included fine-grained deposits in the vertical direction due to the low vertical hydraulic conductivity and the large inelastic specific storage of the fine-grained deposits. This causes a gradual rather than abrupt release of water from inelastic storage, which may be spread across several of the time steps used in the numerical-

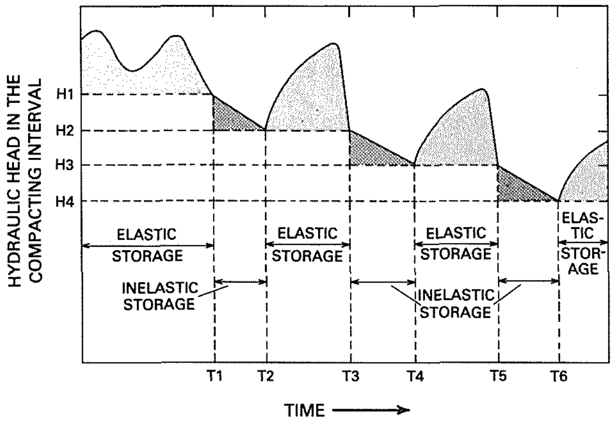


FIG. 3 Relation of storage to hydraulic head in a compacting interval of the aquifer system.

simulation scheme. Thus, the assumption used in this study that all the water is released from inelastic storage within the time step in which the head change occurs is erroneous. However, the simulated values should approximate the observations over several time steps. Second, the change from one storage value to another was done at the beginning of each time step even though the change actually occurred in the previous time step. This meant that small time steps were necessary in the simulation, which increased the computer time and cost of each simulation. Third, the inelastic specific storage is assumed constant, even though laboratory consolidometer tests of small clay samples indicate that the amount of water released from inelastic storage is a function of the applied stress. In addition, the vertical hydraulic conductivity of the compacting deposits, in theory, should decrease with time. However, on the basis of soil consolidation theory, Helm (1977) was able to simulate the total compaction with reasonable results at seven sites in the San Joaquin Valley over periods of decades using constant values for aquifer properties that probably represent average values for the periods selected. The error involved in using constant aquifer properties in this study is thought to be small because the simulation period was only 17 years.

To model ground-water flow through the Central Valley aquifer system, the system was divided into model blocks with horizontal dimensions of 10 km on a side and vertical dimensions that varied from a few hundred meters to a few thousand meters. A maximum of four model layers were simulated in the vertical direction. The uppermost model layer corresponds to the water-table zone, which is generally less than 150 m thick, while the next two model layers correspond to the lower pumped zone, which is generally less than 800 m thick. The lowest model layer includes the aquifer system below the interval of wells. Heterogeneity in the aquifer system caused by variation in the types of sediments was simulated by (1) varying the aquifer properties between model blocks and (2) averaging aquifer properties in each model block to represent the aggregate of the heterogeneity within that block.

The ground-water flow model was calibrated for the period from 1961 through 1975, and the simulations were extended to include the 1976-77 drought. The modeled period was divided into six-month intervals to better simulate the effect of seasonal agricultural pumpage. Recharge was simulated in the autumn-to-spring period. Municipal pumpage was divided evenly, and all of the agricultural pumpage was simulated in the spring-to-autumn period of each year.

The model was calibrated by adjusting the horizontal and vertical hydraulic conductivities, the distribution of recharge to the aquifer system, and the inelastic and elastic specific-storage values so that the computer-simulated hydraulic heads matched (within the limits of the investigation) the observed heads in the aquifer system. Pumpage was not changed.

Results

The simulated water levels at the end of the calibration period (1975) were, on the average, 0.8 m and 3.7 m higher than the observed levels for the water table zone and lower pumped zone, respectively. The standard deviations of the simulated versus observed water levels were 6.7 m and 8.3 m for the two zones.

Initial estimates of elastic and inelastic specific storage were obtained from Poland (1961), Helm (1978), and Ireland and others (1982). The elastic specific storage of the coarse- and fine-grained deposits were estimated as 3×10^{-6} and 1.5×10^{-5} m^{-1}, respectively. The combined average for deposits in the Central Valley was 1×10^{-5} m^{-1}. The inelastic specific storage of the fine-grained deposits was estimated at 1×10^{-3} m^{-1}. In the model simulations, the elastic specific storage was increased by a factor of 2, except in the vicinity of the Los Banos-Kettleman City area (fig. 1), where it did not change. The increase was needed to reduce the simulated water-level fluctuations caused by alternating six-month periods of recharge and discharge. Inelastic storage values in the model simulations changed very little during model calibration.

Initial critical-head values were estimated to be 25 m less than the pre-development water levels of the early 1900's. The 25 m was determined by Holzer (1981) at a few locations in California. Model simulations began in 1961 during a period of major subsidence in several parts of the Central Valley, and water levels in many areas were many meters below the initial estimated critial heads. Thus, in these areas, the critical head was estimated to be the previous low water level, which commonly had occurred during the 1960 pumping season. Critical-head values were adjusted as much as 5 m in several model blocks during calibration. However, water levels in the model blocks correspond to averaged water levels over a 100-km^2 area, and the averaged water levels may be in error by as much as 7 m. On a regional scale, the changes in critical head were, therefore, small.

In general, the calibrated hydraulic properties of the aquifer system and the recharge and discharge changed little from the initial estimate, except for the vertical hydraulic conductivity between the water-table and the lower-pumped zones. The vertical hydraulic conductivity was affected by the number of wells that were perforated or gravel-packed in both zones. However, these values were easily adjusted because the vertical head gradient between the two zones was very sensitive to small changes in the vertical hydraulic conductivity.

The results of simulating land subsidence from the inelastic compaction of fine-grained deposits in the ground-water flow equation compared favorably with the estimated values of land subsidence for the period 1961 through 1977, as shown in table 1. This assumes that the actual land subsidence was caused only by inelastic compaction of fine-grained deposits as a result of ground-water pumpage. The simulated volume of land subsidence resulting from ground-water pumpage was 8 percent less than that estimated for the period 1961-69, 17 percent less than that estimated for the period from 1961 through 1975, and 6 percent less for the period from 1961 through 1977. Estimates of land subsidence made from 1961 through 1977 were based on average rates of subsidence between times of leveling and were prorated to individual years according to extensometer data from wells as reported by Poland and others (1975) and Ireland and others (1982).

TABLE 1.--Comparison of estimated and simulated volumes of land subsidence in the Central Valley for the period from 1961 through 1977

[Volumes in cubic hectometers]

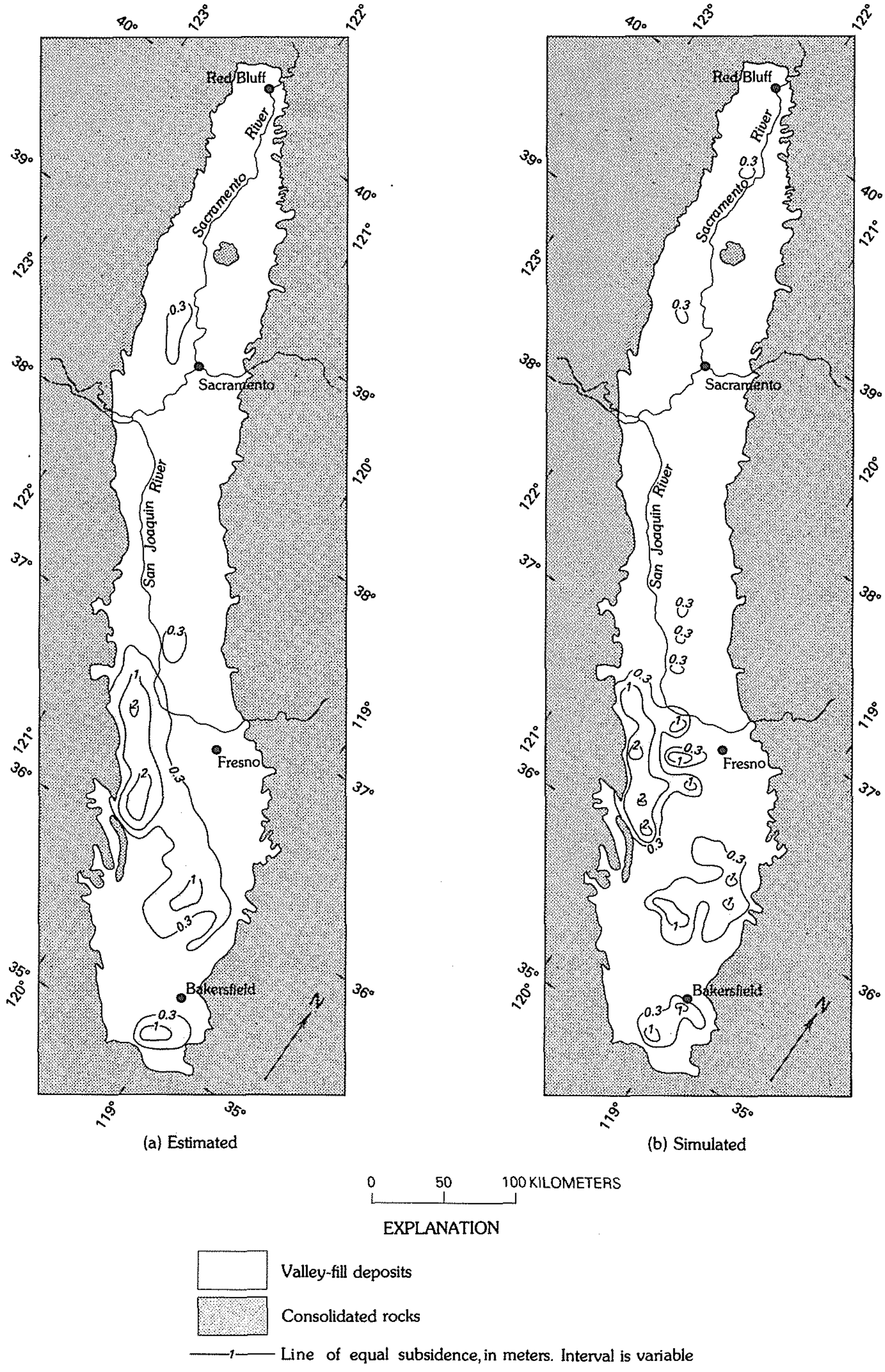
		Land subsidence		
Period	Pumpage[1]	Estimated	Simulated	Percent difference
1961-69	81,000	6,600	6,050	8
1961-75	131,000	8,100	6,700	17
1961-77	153,000	8,950	8,450	6

[1] Estimated pumpage from both the lower pumped zone and the water-table zone.

Most of the land subsidence due to ground-water pumpage has occurred in the San Joaquin Valley. Poland and others (1975) identified three major subsidence areas in the San Joaquin Valley: (1) The Los Banos-Kettleman City area along the southwest side of the Central Valley; (2) the Tulare-Wasco area north of Bakersfield, and (3) the Arvin-Maricopa area south of Bakersfield (fig. 1). The Davis-Zamora area was the only major subsidence area identified as of 1973 in the Sacramento Valley (Lofgren and Ireland, 1973).

Simulated and estimated land subsidence due to ground-water pumpage is shown in figure 4 for the period from 1961 through 1975. Differences between simulated and estimated subsidence can be explained in several ways:

1. Ground-water pumpage from the lower pumped zone was the primary cause of land subsidence in the model simulations. Errors in allocating pumpage to the model grid may cause both the amount and distribution of simulated subsidence to be different than the estimated subsidence.
2. The estimates of subsidence are subject to error because much of the Central Valley has not been releveled since the early 1970's.
3. Simulated subsidence in the model blocks is dependent on the head at which inelastic compaction begins (the critical head). The critical head in the fine-grained lenses of the aquifer system was assumed to be equal to the head in the coarse-grained deposits. This assumption may not be correct because of the time needed for a head change to propagate through the thicker fine-grained lenses in the aquifer system. Errors in estimating the critical head for each model block affect both the distribution and amount of the simulated subsidence, as well as the heads in the lower pumped zone.
4. Model simulations assumed that water was released from the inelastic compaction instantaneously when the water level in a model block declined below the critical head. On the basis of observed data, subsidence continued for several years at some locations after the water levels recovered, but at a slower rate; this is attributed to the time lag for head changes in the coarse-grained deposits to propagate through the fine-grained deposits.



(a) Estimated

(b) Simulated

FIG. 4 Estimated and simulated land subsidence for the period from 1961 through 1975.

5. Simulated subsidence stopped when the water levels started to rise, and simulated inelastic compaction did not begin until the water levels decreased below the previous lowest level. Data collected at a few sites did not show a continued yearly water-level decline, yet the rate of subsidence (although somewhat variable) increased before the previous lowest water level had been reached. The inelastic compaction is again attributed to the time lag for head changes to propagate through the fine-grained deposits. However, some of the subsidence may be elastic, as indicated by negative compaction observed at extensometers at a few sites after the 1977 drought (Ireland and others, 1982).

Relation of Water Released from Inelastic Compaction to Ground-Water Pumpage

Estimates of pumpage were compiled yearly from 1961 through 1977 for most of the Central Valley on the basis of electric-power consumption and pump-efficiency tests (Diamond and Williamson, 1983). In addition, pumpage estimates were divided between the water-table zone and the lower pumped zone. Thus, a comparison between the amount of water released from compaction of the fine-grained deposits and pumpage from the lower-pumped zone for the period 1961 through 1977 was done for each of the major subsiding areas.

The ratio of (1) the volume of water released from inelastic compaction of the fine-grained deposits to (2) the volume of pumpage varied among the four major subsiding areas in the Central Valley. The smallest ratio was in the Arvin-Maricopa area, where about 4 to 5 percent of the water pumped came from inelastic compaction. In contrast, the largest ratio was in the Los Banos-Kettleman City area, where about 35 percent of the pumpage came from inelastic compaction (the proportion was as much as 42 percent from 1961 through 1969, a period of major subsidence).

Differences in the ratio in the lower pumped zone may be explained by (1) variations in the amount, compressibility, and bedding of the fine-grained deposits and (2) variations in the applied stress that tends to compact a deposit.

The Arvin-Maricopa area has the smallest percentage of fine-grained deposits, whereas the Los Banos-Kettlemen City area has the largest. Montmorillonite, which is more susceptible to inelastic compaction than either illite or kaolinite, is the predominant clay in the major subsiding areas in the San Joaquin Valley (Meade, 1967 and 1968). In contrast, kaolinite is the principal clay mineral in soils and alluvium of the northern San Joaquin Valley (Meade, 1967) and in samples collected from three test holes in the Sacramento Valley (R. W. Page, U.S. Geological Survey, written commun., 1983). Thus, the lack of montmorillonite might explain a lesser amount of water released from inelastic compaction in parts of the Central Valley, but it does not explain the differences among the areas in the San Joaquin Valley.

Variation in the bedding of the deposits may also contribute to differences in the amount of water released from inelastic compaction to pumpage. Bull (1975) noted that the interlayering of thin-bedded compressible fine-grained deposits with coarse-grained deposits resulted in a system that compacted more rapidly as a result of increases in applied stress than either (1) mostly coarse-grained deposits or (2) thick fine-grained deposits. In general, the Los Banos-Kettleman City area has more thin-bedded, fine-grained deposits interlayered with coarse-grained deposits than either the Arvin-Maricopa area or the Tulare-Wasco area.

Variations in the amount of the applied stress among the major subsiding areas may also affect the amount of water released from inelastic compaction. These differences may in turn be caused by differences in well construction. Most wells in the Los Banos-Kettleman City area are perforated below the shallow water table, which contains poor quality water. Water is obtained from a thicker interval of the aquifer system in the other areas, where perforated intervals commonly extend from the water table zone into the lower pumped zone. The effect of the long perforated interval is twofold: (1) some of the water pumped from the wells actually comes from the shallower water-table zone and (2) the water levels in both zones are lowered, which decreases the vertical seepage stresses and reduces the amount of inelastic compaction.

Summary

The Central Valley aquifer system consists mostly of deposits of gravel, sand, silt, and clay, of which about half are fine grained. Large volumes of water have been pumped from the aquifer system since the early 1900's, mostly for irrigation. Water levels in the most heavily pumped areas had declined as much as 120 m by 1970. These large water-level declines resulted in the release of about 21,000 hm^3 of water by inelastic compaction of numerous compressible fine-grained deposits, of which about 9,000 hm^3 occurred from 1961 through 1977.

The principal technique used to evaluate the aquifer system was a three-dimensional computer program that solves the basic ground-water flow equation using a finite-difference numerical procedure. The program was modified to incorporate the process of water release from inelastic compaction of the fine-grained deposits, by making storage a function of hydraulic head. A head value (referred to as the critical head) that is related to the maximum effective stress to which the deposits had been previously subjected was assigned to each model block representing a part of the aquifer system. Whenever the simulated hydraulic head decreased below the previous lowest head value, the storage coefficient would change from an elastic value to an inelastic value. Whenever the simulated hydraulic head increased, the storage coefficient would switch back to an elastic value. Land subsidence was calculated by multiplying (1) the hydraulic-head decline below the critical head by (2) the inelastic storage coefficient for each model block. Values of subsidence were accumulated over the simulation period of 1961 through 1977.

Simulated water levels during the model-calibration period of 1961-75 averaged 0.8 and 3.7 m above the observed water levels for the water-table zone and the lower pumped zone, respectively. The standard deviations of those differences were 6.7 and 8.3 m, respectively. The calculated volume of water released by inelastic compaction was about 6 percent less than that estimated for the period 1961-77, and 17 percent less for the calibration period of 1961-75. Differences in simulated and estimated volumes may be attributed to the following causes: (1) The uncertainty inherent in estimating and transferring pumpage information to the model grid; (2) the uncertainty in estimating subsidence; (3) the uncertainty in assuming that the head in the fine-grained deposits is equal to that in the coarse-grained deposits; (4) the uncertainty in assuming that water is released instantaneously from inelastic storage; and (5) the uncertainty in assuming that inelastic compaction did not occur until after computed water levels had declined below previous lows.

The ratio of (1) the volume of water released from inelastic compaction of the fine-grained deposits to (2) pumpage from the lower pumped zone varied among the four major subsiding areas in the Central Valley. These differences may be explained by (1) variations in the amount, compressibility, and bedding of the fine-grained deposits and (2) variations in the applied stress that tends to compact a deposit, which in part can be caused by different well-construction techniques.

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A 3-D NUMERICAL MODEL FOR SUBSIDENCE OF HORIZONTALLY LAYERED FORMATIONS

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Abstract

A numerical method for geothermal subsidence computations in horizontally layered geological formations has been implemented. The method is based on the superposition of fundamental (Green) solutions of ground displacement. In order to achieve greater generality, however, these solutions are obtained by the finite element method rather than by analytical formulas. Thermal and pressure effects for different dimensions of reservoirs and for a variety of geological morphologies are investigated. In particular, the proposed numerical procedure is applied to some production schemes in order to assess the effect of reinjection wells on subsidence.

Introduction

A complete numerical model of phenomena producing subsidence has to take into account the coupling of pressure and thermal field with the state of stress (Lewis et al., 1978). However, for practical purposes and for complex exploitation schemes, it is reasonable to compute subsidence on the basis of a known distribution of pressure and temperature variations in the reservoir (Pinder, 1979). This distribution may be provided by the interpretation of field measurements or by numerical simulations based on more or less sophisticated mathematical models.

Due to the large computational effort required and to the lack of a detailed description of the system, a completely general deformation analysis (e.g. by means of a three-dimensional program) has never been attempted, and a number of very simplified analysis procedures are currently used. One of the most efficient is the nucleus of strain method developed by Geerstma (1976) and Van Opstal (1974) for homogeneous elastic subsoils. An extension to horizontally layered reservoirs was presented by Williamson (1974). His analytical method, however, evaluates only the total volume of subsidence and does not provide the actual surface deformations.

In this paper, the complete set of equations governing subsidence will be presented, extended to the thermal effects due to renjection of cool water into a geothermal reservoir. Successively a more speditive method, valid for horizontally layered formations, will be discussed; it constitutes a new variant of the nucleus of strain method.

Equations governing subsidence

Mass Flow

Equation of mass flow in water dominated geothermal reservoirs is given by:

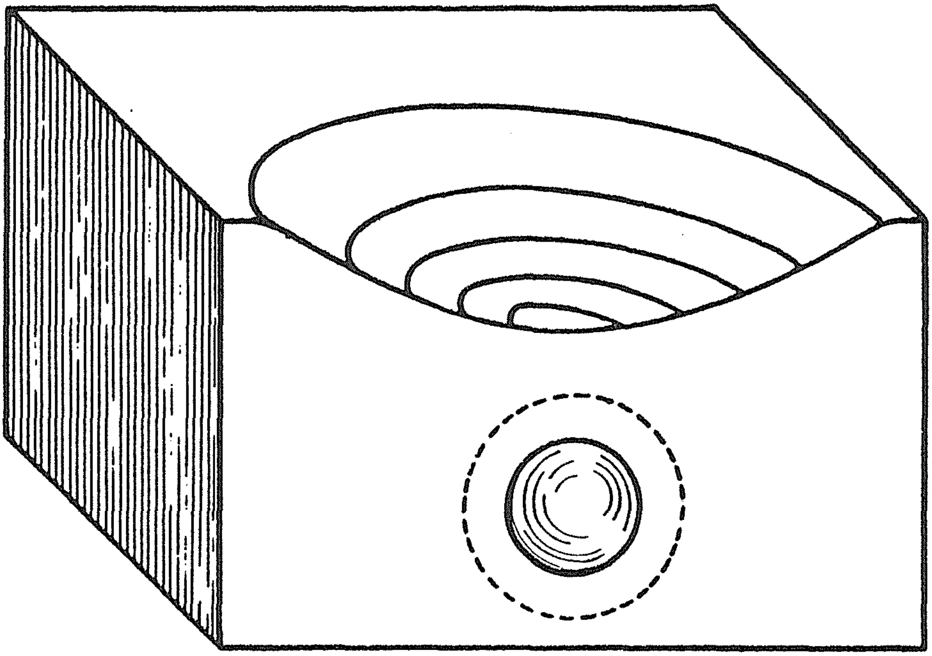


FIGURE 1 - SURFACE SUBSIDENCE
due to the shrinkage of a spherical volume in the subsoil.

$$\nabla \left\{ \frac{kp}{\mu} \nabla \left(p+\rho_W g\, z\right)\right\} = -\left(C-C^*\right)\frac{d\bar{\sigma}'}{dt} + n\left(C_W - C^*\right)\frac{dp}{dt} - n\left(\beta_W-\beta\right)\frac{dT}{dt} \qquad (1)$$

Where Kp is the intrisic permeability μ ,is the viscosity, p is the pressure, ρ_w is the water density, z is the elevation, C the bulk compressibility of rock-mass, C* the bulk compressibility of rock matrix, $\bar{\sigma}'$ is the mean effective stress, t is time variable,n is the porosity, Cw is the water compressibility, β_w is the thermal volumetric expansion coefficient for water and β is the thermal volumetric expansion coefficient for rock. Equation (1) is obtained combining the equation of continuity with Darcy's law.

Heat Flow

Heat balance for a saturated soil element is described by the equation:

$$\left[n\, \rho_W\, c_W + \left(1-n\right)\rho^* c^*\right] \frac{dT}{dt} = -\rho_W\, c_W\, \nabla T \cdot \underline{u} + \nabla \cdot \left(\lambda \nabla T\right) \qquad (2)$$

where c_W ,c^* are the specific heats for water and rock matrix,respectively, $\rho^*$is the rock matrix density,$\underline{u}$ is the vector of fluid velocity, and λ is the thermal conductivity. Equation (2) is obtained by assuming that there is a thermal

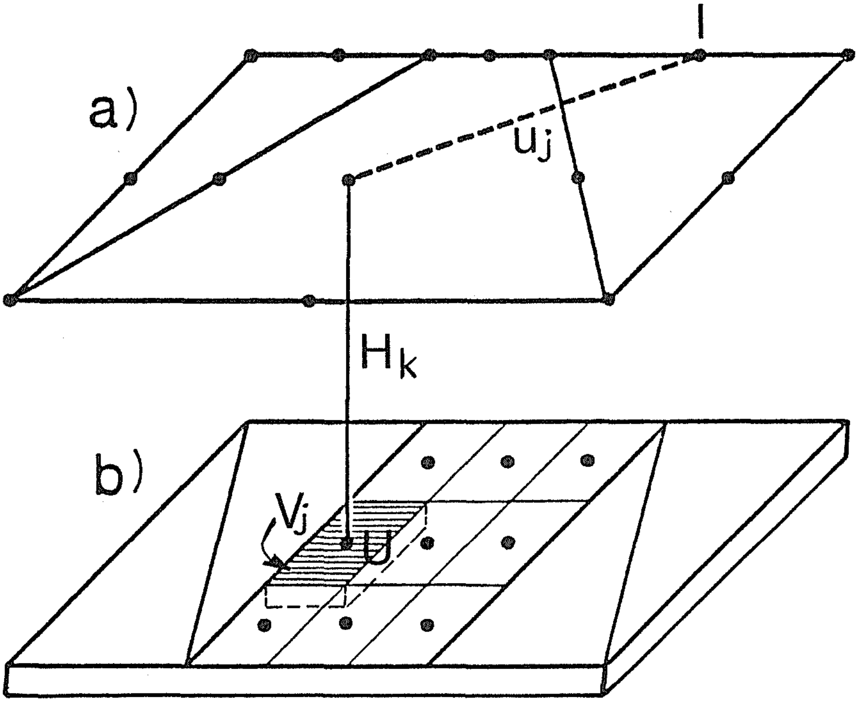


FIGURE 2 -

a) Upper finite element mesh; at its nodes surface displacements are computed.

b) Lower finite element mesh in which are evidenced the volumes corresponding to the 9 points of the Gauss quadrature formula.

equilibrium between the fluid and the solid matrix, thus assigning the same local temperature T to both. This assumption can be accepted when the pore fluid moves with Reynolds numbers lower than 1, as generally occurs in practice.

Stress analysis

The analysis can be carried out in terms of total or effective stresses: either of these approaches can prove to be the most convenient, according to the type of problem to be solved.
When a linear elastic behaviour of the rock is assumed, the stress-strain relationship in terms of effective stresses is

$$\underline{\sigma}' = \underline{\underline{D}} \left(\underline{\varepsilon} - \underline{\varepsilon}_T - \underline{\varepsilon}_P \right) \qquad (3)$$

where $\underline{\underline{D}}$ is the elasticity matrix, and $\underline{\varepsilon}_t$, $\underline{\varepsilon}_p$ are the initial isotropic strains which take into account the effects of temperature and pressure. In addition to the applied forces, the

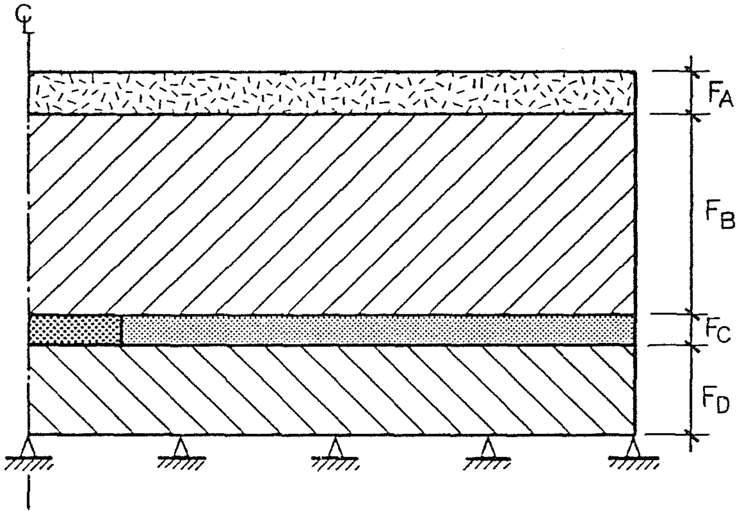


FIGURE 3 - LAYERED SUBSOIL of infinite extent. layers F_A , F_B , F_D are impervious. In the pervious layer F_C a discoidal nucleus of strain is shaded. F_A = 200 m, F_B = 1800 m, F_C = 100 m, F_D = = 1000 m.

analysis must include the body forces corresponding to the pressure gradient.
When a non-linear elastic behaviour is to be taken into account, the stress-strain relationship must be written in an incremental form. The elastic parameters forming the $\underline{\underline{D}}$ matrix should now be updated at each time increment.
Above equations (1), (2) and (3) have been considered by Borsetto,Carradori and Ribacchi (1981) for the implementation of the 2-D planar or axisymmetrical computer code TRAITEME.
This programme takes into account the fully coupling of equations; it utilizes the finite element method for spatial discretization and a generalization of the Crank-Nicholson rule for time integration. For practical purposes,however, in order to reduce computational time and to solve 3-D problems, the computer code SUBTRI has been envisaged; it will be presented in the following.

The nucleus of strain method

This method relates the subsidence to the deformations in the subsoil due to temperature and pressure effects (Figure 1).
The volumetric strain due to a temperature increase of a unit volume can be expressed as

$$\Delta V_T = \beta \Delta T \qquad (4)$$

TABLE I - ELASTIC MODULI OF LAYERS
F_A , F_B , F_C , F_D
for 5 different model problems.

		PROBLEM				
		1	2	3	4	5
LAYER	F_A	.01	.01	.01	1.	.01
	F_B	.1	.25	.5	1.	.25
	F_C	1.	1.	1.	1.	1.
	F_D	1.	1.	1.	1.	10.

The volumetric strain due to a pressure increase of the saturating fluid may be expressed as

$$\Delta V_p = C_v \, \Delta p \qquad\qquad C_v = C - C^* \tag{5}$$

There is an obvious analogy between the effect of pressure and temperature variations. Therefore, for subsidence evaluation, temperature variations can be replaced by equivalent pressure variations: for this reason temperatures are disregarded in the subsequent notation.
Assuming a linear elastic behaviour of the soil, a displacement component D at a surface point I may be computed, using the superposition principle, as

$$D(I) = \int_V \delta(I,U) C_v(U) \, \Delta p \, (U) \, dV \tag{6}$$

where $\delta(I, U)$ is the displacement of I caused by a unit volumetric strain at the reservoir point U. Computational advantages accrue if δ does not depend on the two particular points, but only on some relationship between their coordinates, as

$$\delta(I,U) = \delta(u,h) = \delta_h(u) \tag{7}$$

where h is the depth of U and u is the distance of U from the vertical axis passing through I. Such a situation occurs in horizontally layered soils. Analytical expressions of $\delta_h(u)$ are available for homogeneous soils (Van Opstal, 1974).
The fundamental solution $\delta_h(u)$ for more complex geological stratigraphies can be computed by a numerical procedure, such as the finite element method. In fact $\delta_h(u)$ can closely be approximated by solving an axisymmetric problem (as shown in the next paragraph) with an initial unit volumetric strain in a finite volume V_n (nucleus of strain). Of course, the maximum diameter of the volume V_n must be small with respect to the average depth H_k. The same displacement solution $\delta_{H_k}(u)$ is ap-

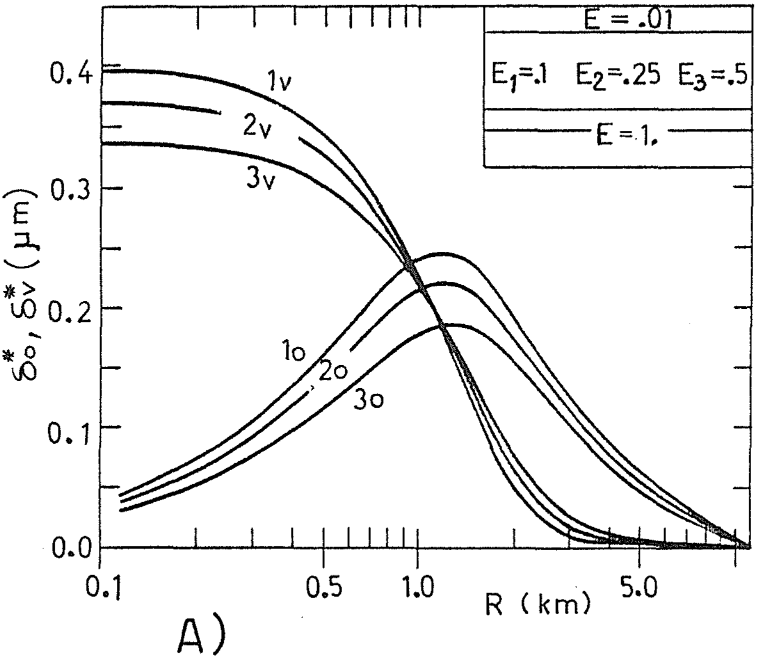


FIGURE 4 - SURFACE DISPLACEMENTS FOR A NUCLEUS OF STRAIN with radius = 34 m and uniform volumetric strain = $-2.4\text{x}10^{-5}$. Poisson's ratio is 0.2. δ_o^* = centripetal horizontal displacement. δ_v^* = downward vertical displacement. Numbering of curves refers to the different cases in Table I.

proximately valid for any volume V_j of similar magnitude and located at the same depth, provided that the foundamental solution is multiplied by the ratio V_j / V_n . The evaluation of the integral in Equation (6) is therefore reduced to the summation

$$D(I) = \sum_{k=1}^{K} \sum_{j=1}^{J} \delta_{H_k}(uj) \; C_{vj} \; \Delta p_j \; (V_j/V_n) \qquad (8)$$

where (J X K) is the number of subdomains into which the reservoir is divided.

A convenient way of operating is to use the interpolation properties of the finite element method in order to compute the contribution to the right-hand side of Equation (8), especially when pressure and temperature fields are previously calculated by the same method. In this case, V_j is the volume corresponding to the integration weight of a quadrature formula. Moreover, displacement D(I) may be evaluated at the nodes of a finite element mesh, in order to utilize the differentiation

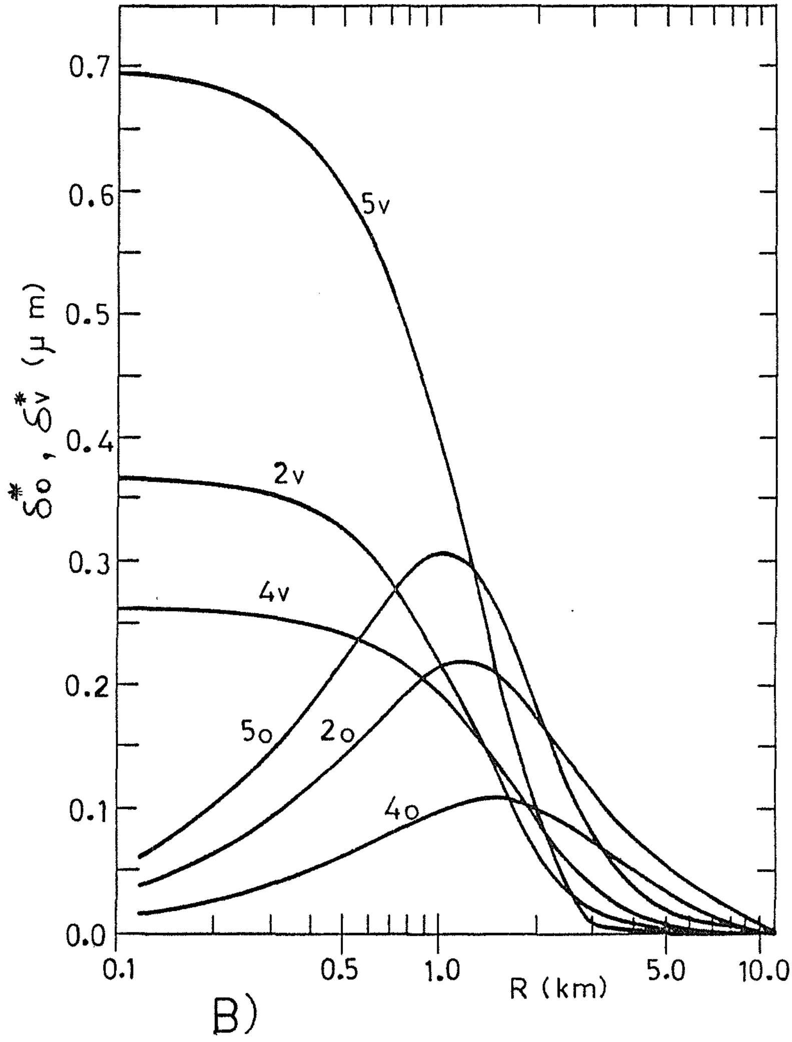


FIGURE 4B (Continued)

properties of the interpolating polynomials in the calculation of strains, tilts and curvatures(Figure 2).

Computation of nucleus fundamental solutions

In this section fundamental solutions are computed for the geological formation of Figure 3. The fundamental solution, used in the following, is that due to the uniform shrinkage of a discoidal volume located on the axis of symmetry. The formation is modelled with axisymmetric finite elements up to a radial distance of 11 km. Solutions are computed for different model problemsin order to point out the influence of the stiffness of the various layers. A summary of the analysed problems is given in Table I. Surface displacements for the first three cases are shown in Figure 4(a).

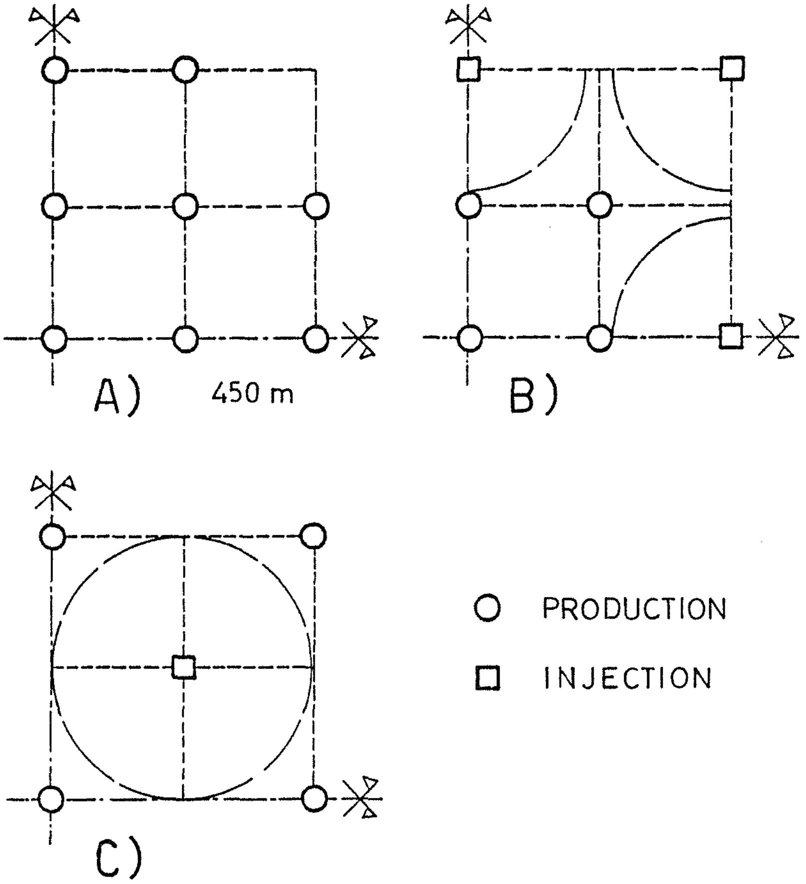


FIGURE 5 - PLANT WIEW OF THREE DIFFERENT EXPLOITATION SCHEMES
Only a quadrant is represented. Circles around injection wells show the cooled region after 11 years (5B) and 7 years (5C).

The problems correspond to three different stiffness values of the cap formation F_B. The curves for Problem 2 are again shown in Figure 4(b) to be compared with the results obtained for uniform subsoil and for a stiff formation underlying the reservoir. In this case, the results appear to be more sensitive to the varying stiffness values.

Subsidence for some productin schemes in geothermal reservoirs

Three different production schemes in hot-water reservoirs are studied for the geological stratigraphy of Figure 3 and for the second set of elastic moduli in Table I. The selected schemes are shown in Figure 5, where only a quadrant is considered be-

TABLE II - RESERVOIR PROPERTIES

PERMEABILITY $k_p = 5x10^{-14}\ m^2$

VOLUMETRIC COMPACTION COEFFICIENT $C_v = 1.75x10^{-4} MPa^{-1}$

TEMPERATURE T = 200°C

WATER VISCOSITY AT 200°C $\mu = 0.15x10^{-9} MPa\ x\ s$

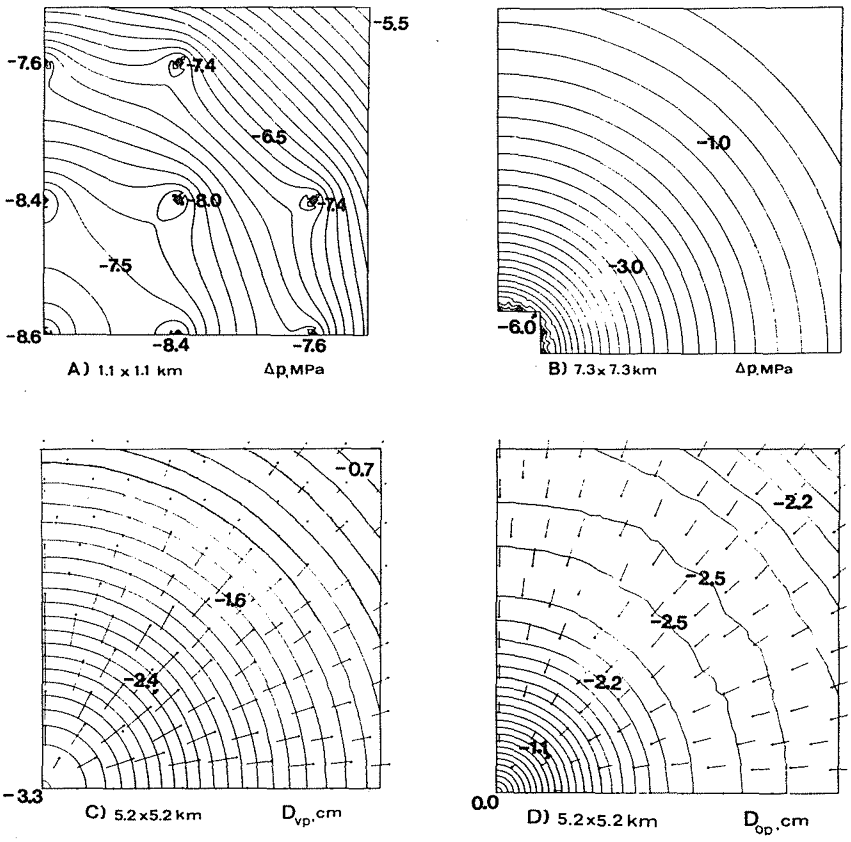
VOLUMETRIC THERMAL EXPANSION COEFFICIENT $\beta = 2.4x10^{-5}\ °C^{-1}$

WELLBORE RADIUS a = 0.15 m

cause of symmetry. A total production rate Q = $63x10^{-2}$ m^3 /s, equally supplied by production wells, is assumed. For the schemes of Figures 5(b) and 5(c), where reinjection wells-as in the secondary oil recovery schemes-are present, the total amount of water producted is injected at the same rate for each well. A constant pressure boundary is located at a distance r_e of about 9 km from the centre .
Pressure variations are evaluated by superimposing the stationary solutions Δp for a single well producing the flow rate q:

$$\Delta p = p_a\left(1-\frac{\ln(r/a)}{\ln(r_e/a)}\right)\ ;\quad p_a = q\ln\left(r_e/a\right)\frac{\mu}{2\pi K F_c} \qquad (9)$$

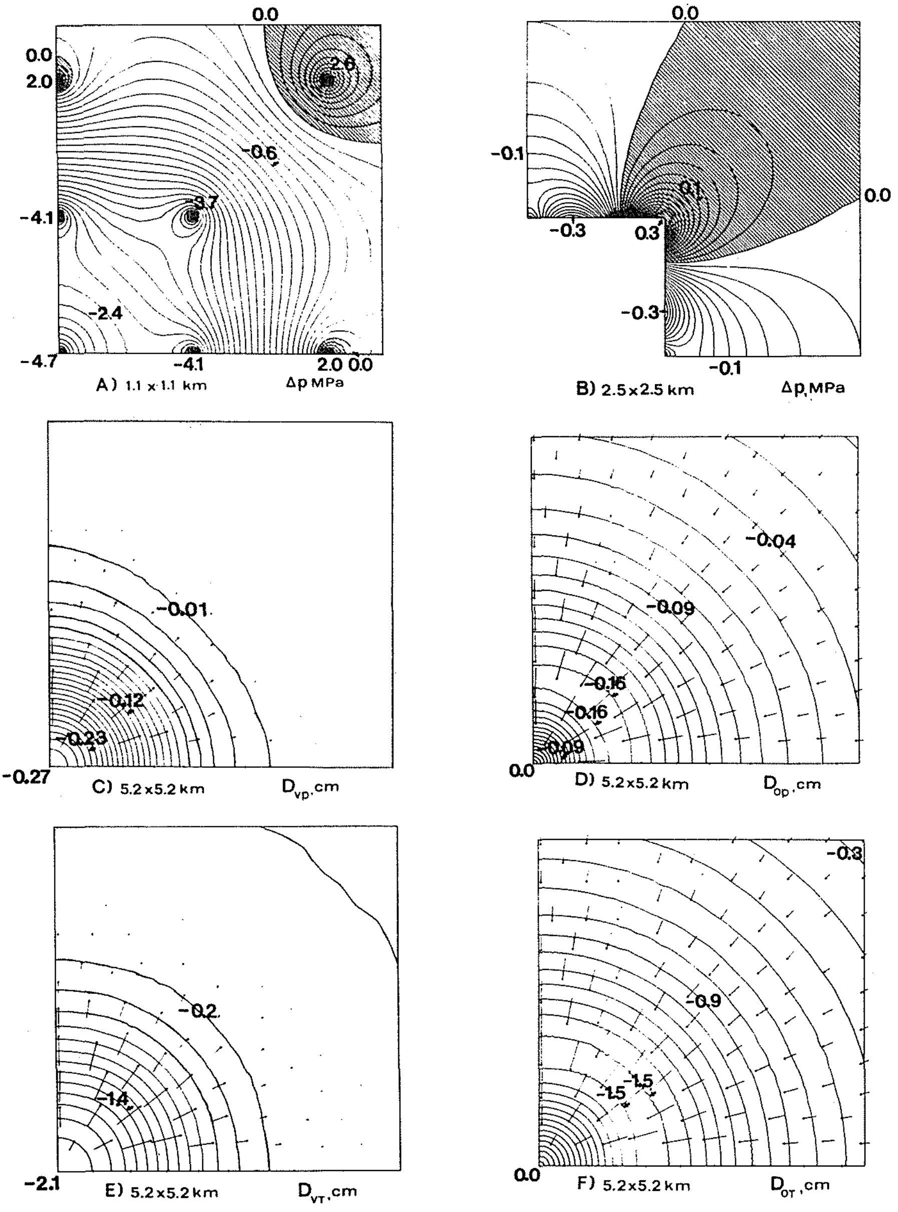
For simplicity, the viscosity of the fluid is assumed constant.
Actually, pressure transients are very short, and may be disregarded.
Pressure for the scheme of Figure 5(a) is plotted in Figures 6(a), 6(b) and main subsidence effects at the surface are shown in Figures 6(c) and 6(d). Surface effects are nearly axisymmetric, owing to the regular disposition and the small distance between wells, in comparison with the depth. Stationary pressure distributions for the second and third scheme (Figures 5(b) and 5(c)) are reported in Figures 7(a), 7(b), 8(a), 8(b) and the corresponding displacements are presented in Figures 7(c), 7(d), 8(c), 8(d). Pressure variations and consequent subsidence are lessened with respect to the first scheme (Figure 5(a)); moreover, some upward displacement arises (Figure 8(c)). As expected, reinjection pressures have a positive effect on subsidence, due to poroelastic phenomena.
In the last two schemes (Figures 5(b) and 5(c)), however, reinjection determines the growth of two cooled regions, which may be considered circular with a reasonable approximation. The following cooling radii, R_B= 400 m and R_C= 450 m, are considered (Figure 5), which correspond approximately to 11 and 7 years of injection. In Figures 7(e), 7(f), 8(e) and 8(f), thermal subsidence is shown for a reference temperature drop



FIGURES 6 - PRESSURE VARIATIONS AND DISPLACEMENTS FOR PRODUCTION SCHEME IN FIG. 5A
Contours of pressures Δp: 6A, 6B. Contours of vertical displacements D_{Vp} and vectors of tilt due to pressure: 6C. Maximum tilt = = 5.5 μrad. Contours of horizontal displacement D_{Op} due to pressure: 6D. Overpressures, upward and centrifugal displacements are positive.

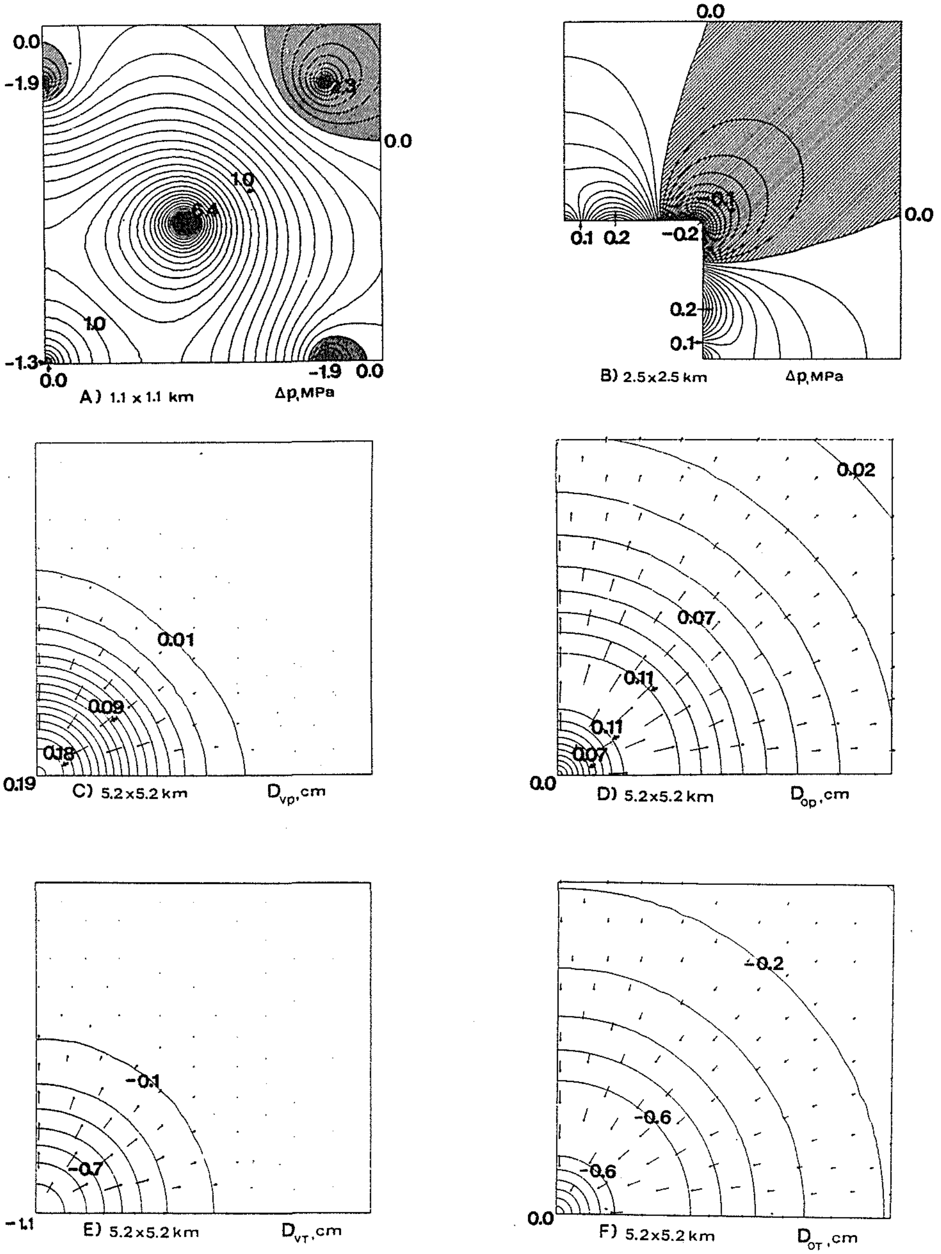
ΔT = -100°C that yields values comparable to those of Figure 6(c), 6(d). It the injection is interrupted, pressure becomes uniform quite rapidly, while the aforementioned thermal subsidence remains unchanged for a long time.

A more correct evaluation of pressure field takes into account temperature effects on viscosity. This computaion-not reported here-has been made for the case of Figure 5(c). A noticeable increase in pressure with respect to the previous distribution in Figure 8(a) occurs only in the cooled region, and does not reduce the thermal subsidence effect by more than 20%, however.



FIGURES 7 - PRESSURE VARIATIONS AND DISPLACEMENTS FOR PRODUCTION SCHEME IN FIG. 5B

Contours of pressures Δp: 7A, 7B. Contours of vertical displacements D_{Vp}, D_{VT}, and vectors of tilt due to pressure (7C) and to temperature (7E). Maximum tilt due to pressure - 1.46 μrad. Maximum tilt due to temperature = 9.7 μrad. Contours of horizontal displacements D_{Op}, D_{OT} due to pressure (7D) and to temperature (7F).



FIGURES 8 - PRESSURE VARIATIONS AND DISPLACEMENTS FOR PRODUCTION SCHEME IN FIG. 5C.

Contours of pressure Δp: 8A, 8B. Contours of vertical displacements D_{Vp}, D_{VT} and vectors of tilt due to pressure (8C) and to temperature (8E). Maximum tilt due to pressure = 1.97 μrad. Maximum tilt due to temperature = 5.6 μrad. Contours of horizontal displacements D_{Op}, D_{OT}, due to pressure (8D) and to temperature (8F).

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NUMERICAL SIMULATION OF SUBSIDENCE DUE TO PUMPING WITH HYSTERESIS EFFECT INCLUDED

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Abstract

Highly compressible impermeable, materials have in general different compressibility indexes in the effective stress ranges above and below the preconsolidation stress; this is traduced in different specific storage coefficients.

I. Herrera and coworkers (see references) developped a numerical method to simulate the flow in a system of aquifers separated by aquitards which includes a boundary solution for the flow in the confining layers and which proved to be accurate and more efficient than a numerical integration over a discretization grid of the aquitard. Yet, it cannot be directly applied to cyclic head variations because of the afore mentioned hysteresis effect.

An approximate solution is presented here which extends the boundary method to such situations. The solution was tested against finite differences results, which in general take much more computational effort and storage allocation, obtaining very good approximation in all situations tested. It was also tested against field measurements of subsidence with equally good coincidence.

Introduction

It is a well known property of clayey deposits to have a larger vertical compressibility index when the effective stress (total minus pore water pressure) acting on the solid structure is above the preconsolidation stress (maximum preconsolidation stress occurred in the history of the material) than when it is below it. This larger compressibility corresponds to the so called "virgin" state of the deposit in which its internal structure is weak to external pressures and deforms rapidly; when the effective pressure is relieved, the structure does not recover its original state but yet behaves elastically to changes in stress with a much lower strain-stress ratio; when the preconsolidation stress in surpassed, the material regains its higher compressibility. The compressibility of the material in the virgin state is in general one to two orders of magnitude larger than in the preconsolidated state. This is shown schematically in Fig 1 with typical results from a drained, vertical consolidation test.

This behaviour translates into geohydrological terms as a larger specific storage coefficient when the pore water pressure drops below the hystorical minimum (which, keeping total pressure constant, corresponds to a maximum effective vertical stress) than when it is above it. In aquifers which are in contact with compressible, low permeability deposits there is an important amount of water released to the aquifer from these deposits when the pressure drops due to pumping or to a natural decline in recharge to the aquifer. This water comes from the destroyed internal structure of the fine grain materials and does not return back to it as easily as it went out. This behaviour has to be taken into account in order to explain and predict water flow and land subsidence in aquifer systems consisting of

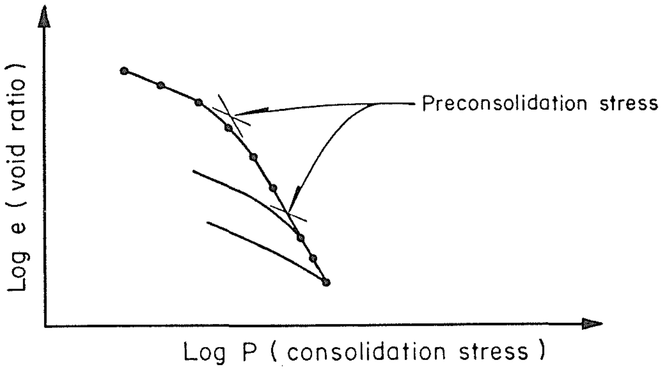

Fig 1 Typical results from drained consolidation tests

permeable and impermeable layers and subject to alternate periods of head drawdown and recovery. Its importance and consequences are remarkably well described by Lofgren (1978) for the aquifer below Pixley, Calif. where very well designed field measurements have been carried on.

To compute aquifer behaviour its now a universal practice to use two dimensional numerical models discretizing the plan extension of the permeable layers; this is done so because in general the thickness of these layers is small compared to its horizontal dimensions. The lower permeability of the confining layers (aquitards) makes it possible to assume that the induced flow in them is perpendicular to the flow in the aquifers; in this way, an obvious extension of the horizontal discretization is that of a vertical, one dimensional grid to compute the flow in the aquitards, building a quasi three dimensional numerical model. With such a model, the non linearity introduced by the above described variability of the specific storage coefficient can be easily taken into account by means of iterative solution methods. Nevertheless, for relatively thick aquitards, the number of grid points may need to be very large in order to accurately compute the flow.

A method has been developped recently (Herrera et al., 1973, 1977, 1980, 1982) which avoids the vertical discretization of the confining layers by computing the flow directly at their boundary with the aquifers and adding if to the mass balance of the latter, thus reducing the dimensionality of the problem and the computational and storage effort. This is done by solving the one dimensional flow equation in the aquitard (see Fig 2):

$$K^1 \partial^2 H/\partial Z^2 = S^1_s \; \partial H/\partial t \qquad (1\ a)$$

with boundary conditions:

$$H(o,t) = H_1(t);\ H(b,t) = H_2(t) \qquad (1\ b)$$

and initial condition:

$$H(Z,0) = \text{constant} \qquad (1\ c)$$

where: $H = p/\gamma + Z$ - piezometric head; p - pore water pressure; γ - specific weight of water; Z - vertical coordinate; K^1 - aquitard hydraulic con-

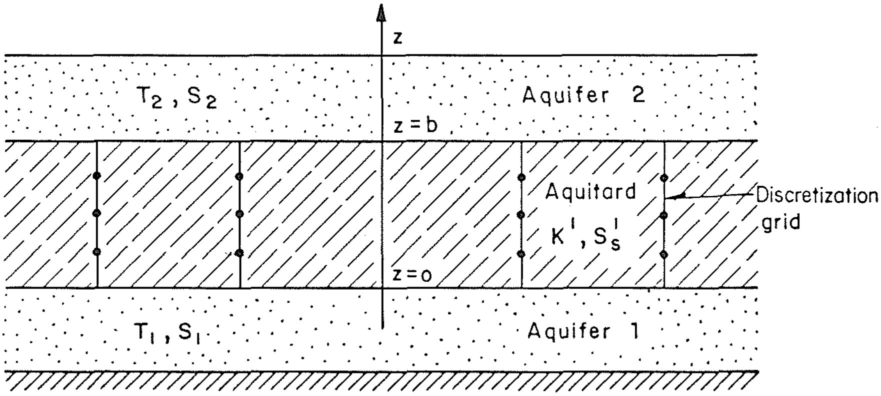

Fig 2 Layered system considered

ductivity - S_s^1 - aquitard specific storage; b - aquitard thickness; t - time; H_1 - piezometric head of the lower aquifer; H_2 - piezometric head of the upper aquifer.

In their solution K^1, S_s^1 are assumed to be constant from Z=o to Z=b. Eqs (1) are solved and the flow at the boundary with each aquifer is added to its mass balance; for aquifer 1 this is expressed by:

$$T_1(\partial^2 H_1/\partial X^2+\partial^2 H_1/\partial Y^2)-(K^1/b)\int_o^t \partial H_1/\partial t\Big|_\tau \; f\left[\beta(t-\tau)\right]d\tau$$

$$+(K^1/b)\int_o^t \partial H_2/\partial t\Big|_\tau \; h\left[\beta(t-\tau)\right]d\tau+Q_1 = S_1\partial H_1/\partial t \qquad (2)$$

and a similar equation for the upper aquifer. In Eq (2) Q_1 is an external unit source and the unit response functions are:

$$f(\beta t) = 1+2\sum_{n=1}^{\infty} e^{-n^2\pi^2\beta t}$$

$$h(\beta t) = 1+2\sum_{n=1}^{\infty} (-1)^n e^{-n^2\pi^2\beta t}$$

$$\beta = K^1/(S_s^1 b^2)$$

where f is called the memory function and varies rapidly from infinity at the origin, approaching the value f=1 around βt = 0.4; h is called the influence function and varies from h=o at the origin, approaching h=1 for βt=0.4; it is for this dimensionless time that the flow becomes established in the aquitard for a unit head difference between the two aquifers and no water is then realeased from aquitard's storage; on the contrary, for very short dimensionless times most of the water is realeased from storage. This

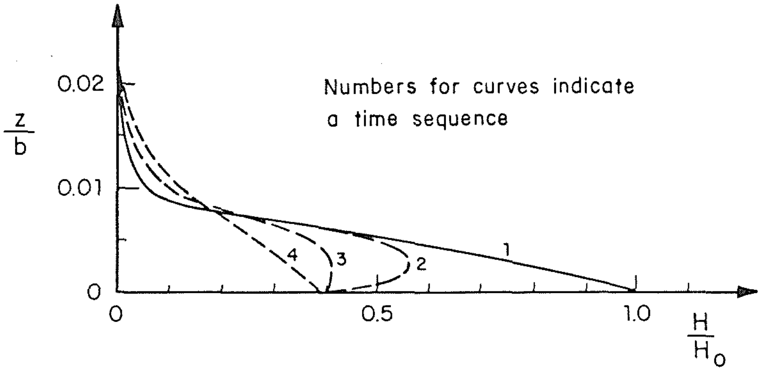

Fig 3 Head profiles after a sudden recovery

behaviour of the unit response functions helps also in fixing the condition of accuracy in a discretization of the aquitard if a numerical scheme is used to compute the vertical flow; any such scheme assumes linear, or nearly so, pressure distribution between grid points, so that for each aquitard subdivision its local dimensionless time interval should be equal or larger than 0.4: $K^1\Delta t/(S_s^1\ \Delta Z^2)\geq 0.4$ or $\Delta Z\leq|K^1\Delta t/(0.4\ S_s^1)|^{1/2}$. This condition is very near to that of numerical stability of an explicit finite difference scheme: $\Delta Z\leq(K^1\Delta t/S_s^1)^{1/2}$ and may become a serious limitation even for implicit schemes since the value of S_s^1 may change by two orders of magnitude, as was pointed out above.

The convolutions in Eq (2) may be easily evaluated numerically without the need of storing the head history at the aquifer and then incorporated to the system of equations to be solved for the aquifers (Herrera and Yates, 1977). Yet, the method cannot be applied directly when water levels alternately raise and go down because then S_s^1 will be variable in depth and time making the problem non linear.

Analysis of pressure profiles

In an attempt to gain insight into the problem when S_s^1 is variable, vertical head distributions were computed using finite differences with very fine point nets for different head hystories, ratios of head recovery to previous drawdown and viceversa, ratios of recovery times to drawdown times and viceversa; two simple conclusions were drawn from the evolution of the head profiles:

1) If after a period of drawdown a recovery takes place, the head profiles have evolutions similar to that shown in Fig 3; comparing these profiles with the preconsolidation head profile, one observes that most of them have heads larger than the latter (corresponding to the lower specific storage coefficient and which, for ease in the exposition, it will be called hereafter "state 2") and for several periods of integration time; in the long run, the upper part of the head profile should merge into the zone of lower than preconsolidation heads (large specific storage coefficient and named hereafter "state 1") but leaving always the lower part, which determines the amount of water interchange between aquitard and aquifer, in state 2.

2) If after a period of recovery, a larger than previous head drawdown is induced at the aquifer, the resulting head profile has all values lower

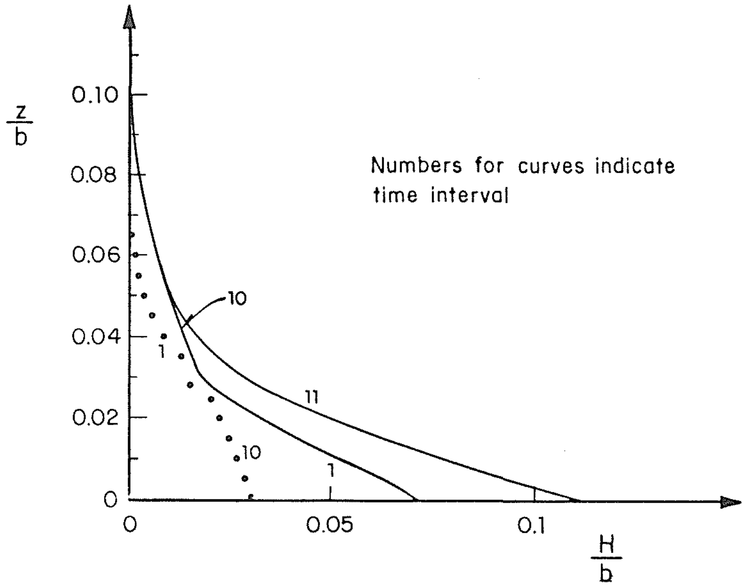

Fig 4 Head profile going from recovery (10) to drawdown (11)

than the preconsolidation head profile (state 1) but has had an evolution through state 2 (see Fig 4).

Hypothesis and solution

These two observations lead to adventure the hypothesis that the specific storage coefficient could be considered to be mainly determined by the stress state of the region of the confining layer nearer to the aquifer, for computational purposes. In the case of head recovery the storage coefficient would be the one corresponding to the higher than preconsolidation head (state 2) and should remain there provided the head at the boundary didn't fall below the minimum of the head history; if the head should drop below that value after a previous recovery, then part of the depletion would take place under state 2 and part of it under state 1. These assumptions avoid the need to take the variation of S_s^1 with the vertical coordinate in the analysis, but still leaves its time dependence which in its turn is dependent on the stress state of the boundary.

The equations to solve are still Eqs (1) with S_s^1 a function of time; Eq (1a) is non linear, but may be solved considering it piecewise linear between finite time intervals and then making the interval tend towards zero. The solution is very similar to that expressed in Eq (2) for boundary flows, with the difference that the argument of the unit response functions f and h is now it self also an integral: instead of $\beta(t-\tau)$ one gets as the argument $\int_{\tau}^{t} \beta(\tau_1)d\tau_1$

Numerical integration

The change in the solution does not introduce any serious difference in the numerical integration previously proposed (Herrera and Yates, 1977) and is

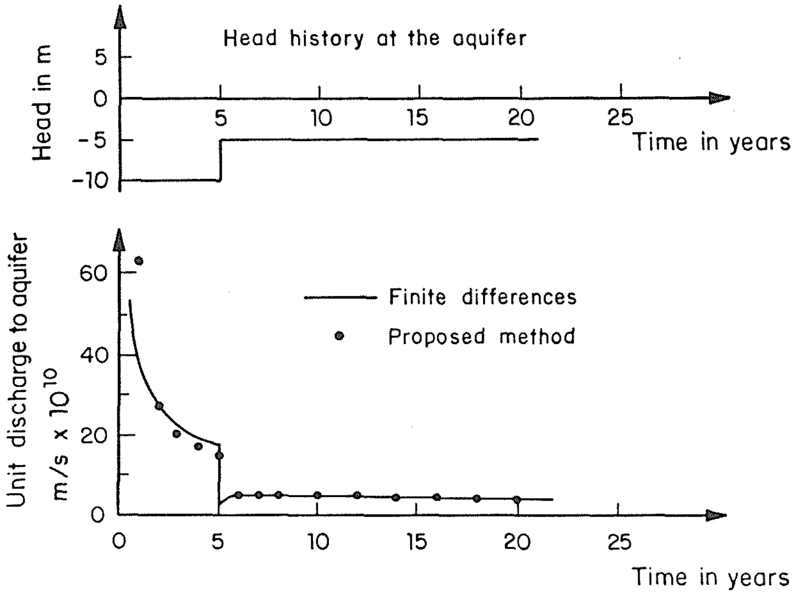

Fig 5 Comparison of results. Head recovery

briefly described in the Appendix; the main change is due to the fact that the storage coefficient is not known in advance and an iterative procedure has to be used to solve the aquifer system of equations. Also in the merging profile case (transition from state 2 to state 1) the integration time interval and the head decline have to be split in two parts, the first one having parameters pertaining to state 2 and the second one to state 1; the time corresponding to state 2 is obtained assuming a larger speed of response for this state than in state 1, a factor equal to the square root of the specific storage coefficients ratio (that of state 2 divided by that of state 1). In the numerical solution, as in the solution of the differential Eq (1), the response is considered piece wise linear with sudden change in material properties (see Appendix).

Model verification

Since there is no exact solution to the problem with a variable storage coefficient (with both vertical coordinate and time), the best approximation that may be obtained is that of a numerical integration over a vertical point net discretizing the aquitard; as pointed out above, the non linearity of the equation, forces the solution scheme to be iterative. The proposed method was tested against finite difference solutions for the unit interchange discharge between aquitard and aquifer. The head history was prescribed at the boundaries with rather extreme and abrupt changes in order to test the method stability, and with conditions receding from the assumed basic hypothesis.

Two examples of result comparison are shown in Figs 5 and 6: The first one corresponds to a long head recovery period after drawdown, a condition which, as may be recalled from Fig 3, brings a large part of the head pro-

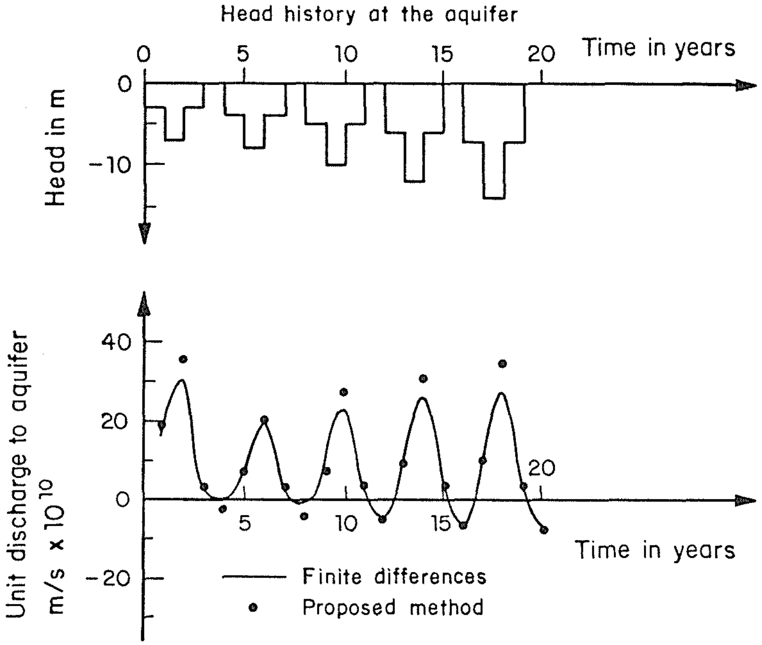

Fig 6 Comparison of results. Cyclic drawdown

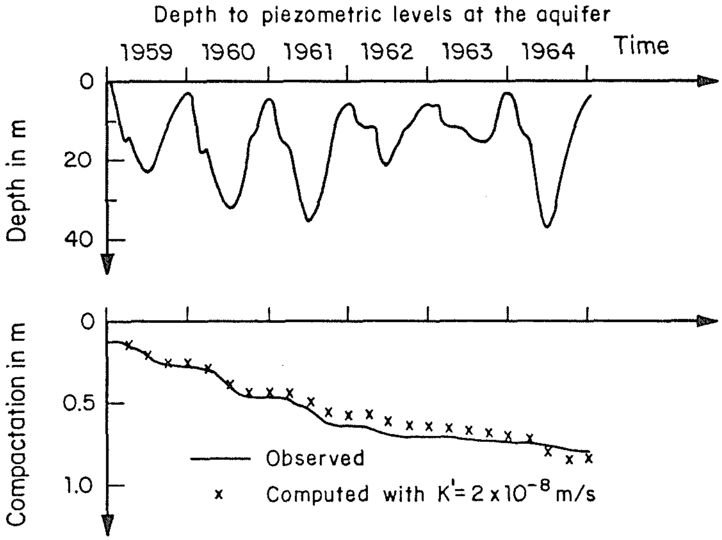

Fig 7 Observed and computed subsidence. Aquifer at Pixley (Lofgren ,1978)

file out of state 2 into state 1, contrary to the method hypothesis; the good agreement with finite differences results indicates that the lower part of the head profile controls the flow, as presupposed. The second case (Fig 6) refers to a cyclic head change with growing minimum heads; the agreement there is also good with finite differences results except in the peaks; this might be due to the rounding off tendency of the implicit numerical scheme needed in the finite difference solution.

After testing the method in these extreme conditions it is not surprising that it fits results for more slowly varying conditions as are real boundary conditions. What is probably more reassuring is that if fits also field measurements. The method was used to compute soil consolidation registered in the aquitard overlying the aquifer below Pixley, Calif. as reported by Lofgren (1978). He presents a six year correlation among piezometric levels in the confined aquifer and compactation (consolidation) of the confining layer of 100 m thickness. Taking the water well hydrographs as boundary heads, consolidation was computed with the proposed method using the reported aquitard characteristics; only the vertical permeability had to be assumed (since no value was reported) to reproduce the observed consolidation evolution with good agreement as shown in Fig 7. An important characteristics of the aquitard at the Pixley location is the large ratio of elastic to virgin deformation modulus which causes that head variations within the elastic range (over preconsolidation head) have little effect on soil consolidation or expansion.

Conclusions

The proposed method to compute flow and land subsidence due to pumping with cyclic head variations proved to give good results when compared to the more computing time and storage consuming discretization methods. Although based on a partially true hypothesis the method is supported by the formation of consolidated layer near to the aquifer controlling largely the vertical flow interchange between aquifer and aquitard, be it in recovery head periods or in drawdowns after recoveries. This was confirmed testing the method as described for extreme head variations and in reproducing some local field consolidation measurements.

Thus, the method is recommended to be used in the evaluation of flow and land subsidence due to pumping in aquifers in contact with compressible layers and with cyclic head variations, or to evaluate pump suspension policies in those aquifers. For this purpose the solution has to be coupled with a two dimensional model for the permeable layers as in Eq (2). The solution of the coupled system of equations has to be iterative due to the their nonlinearity. The iterative procedure has also been tested showing to be rapidly convergent.

Appendix. Numerical evaluation of convolutions

Following Herrera and Yates' (1977) development with the new solution to Eqs (1) with S_s^1 a function of time, the aquitard unit flow contribution to aquifer 1 is giben by:

$$q_1 = \frac{K^1}{b^1} \int_o^t (\partial H/\partial t)_\tau \; f\left(\int_\tau^t \beta d\tau_1\right) d\tau - \frac{K^1}{b^1} \int_o^t (\partial H/\partial t)_\tau \; h\left(\int_\tau^t \beta d\tau_1\right) d\tau \qquad (3)$$

Consider the first integral only and substitute the expression for f (Eqs. 2):

$$\int_{o}^{t} (\partial H/\partial t)_{\tau} \ (1+2 \sum_{n=1}^{\infty} e^{-n^2\pi^2\int_{\tau}^{t} \beta d\tau_1}) d\tau$$

The first term can be evaluated easily between time steps taking a mean value for $(\partial H/\partial t)_{\tau}$ in each interval. Now, write the second term for the next time:

$$2 \sum_{n=1}^{\infty} J_{n,\ k+1} = 2 \sum_{n=1}^{\infty} \int_{o}^{t+\Delta t} (\partial H/\partial t)_{\tau} \ e^{-n^2\pi^2\int_{\tau}^{t+\Delta t} \beta d\tau_1} \ d\tau_1$$

$$= 2 \sum_{n=1}^{\infty} \int_{o}^{t} (\partial H/\partial t)_{\tau} \ e^{-n^2\pi^2\int_{\tau}^{t+\Delta t} \beta d\tau_1} \ d\tau + 2 \sum_{n=1}^{\infty} \int_{t}^{t+\Delta t} (\partial H/\partial t)_{\tau} \ e^{-n^2\pi^2\int_{\tau}^{t+\Delta t} \beta d\tau_1} \ d\tau$$

Considering β constant during the time interval:

$$2 \sum_{n=1}^{\infty} J_{n,\ k+1} = 2 \sum_{n=1}^{\infty} e^{-n^2\pi^2\beta_{k+1} \ \Delta t} \int_{o}^{t} (\partial H/\partial t)_{\tau} \ e^{-n^2\pi^2\int_{\tau}^{t} \beta d\tau_1} \ d\tau$$

$$+ \ 2(\partial H/\partial t)_{k+1} \sum_{n=1}^{\infty} (1 - e^{-n^2\pi^2\beta_{k+1} \ \Delta t})/(n^2\pi^2\beta_{k+1}) \qquad (4)$$

This provides a recurrence formula since:

$$J_{n,\ k} = \int_{o}^{t} (\partial H/\partial t)_{\tau} \ e^{-n^2\pi^2\int_{\tau}^{t} \beta d\tau_1} \ d\tau$$

is known form the preceding time. Its first value (after the first time interval) is:

$$J_{n,\ 1} = (\partial H/\partial t)_1 \ (1 - e^{-n^2\pi^2\beta_1\Delta t})/(n^2\pi^2\beta_1)$$

The method to compute the infinite series after the mentioned authors is, for large and intermediate times ($\beta\Delta t > 0.005$), to truncate the series to a number of finite number of terms and translate the rest of them to the origin to maintain total yield and, for shorter dimensionless times, to use the same procedure with a change in the kernel argument (Chen and Herrera, 1982). An equivalent to this last approach was used here, modified to widen the applicability limits of the computations to account for the great variability of the specific storage coefficient with time. The method is based on the approximation of the series by an integral which in its turn is evaluated numerically by quadrature formulae. This will be described elsewhere.

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ANALYSIS OF LAND SUBSIDENCE DUE TO WITHDRAWAL OF GROUNDWATER IN THE NOBI PLAIN

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Abstract

Land subsidence of the Nobi plain in Japan was mainly caused by lowering of the piezometric heads of groundwater due to the withdrawal of it.

Allowable groundwater levels and yields for preventing land subsidence were studied with one-dimensional and three-dimensional finite element models of the Quaternary sediments.

Land subsidence for several future plans of groundwater use was predicted by the aid of these two models. The predicted results were checked with the observed data. Consequently it was cleared that the relation between predicted land subsidence and withdrawal of groundwater was slightly different from the observed one. This discrepancy could be solved by taking account of the movement of the Tertiary strata, which existed under the model, with change of groundwater head.

History of land subsidence in the Nobi plain

The alluvial Nobi plain, which has an area of 1300 km^2, is located in the central part of Japan as shown in Fig.1.

The history of ground subsidence in this area is shown in Fig.2 by the use of three bench marks.

The yearly rates of subsidence in this area were 1.4-1.8 mm before 1925, 2-5 mm during the period from 1950 to 1960, 20-40 mm and more than 100 mm at severe subsided area during the period from 1960 to 1973.

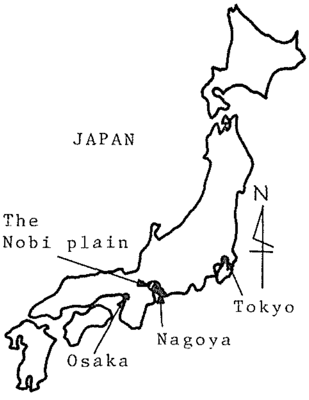

Fig.1 The Nobi plain

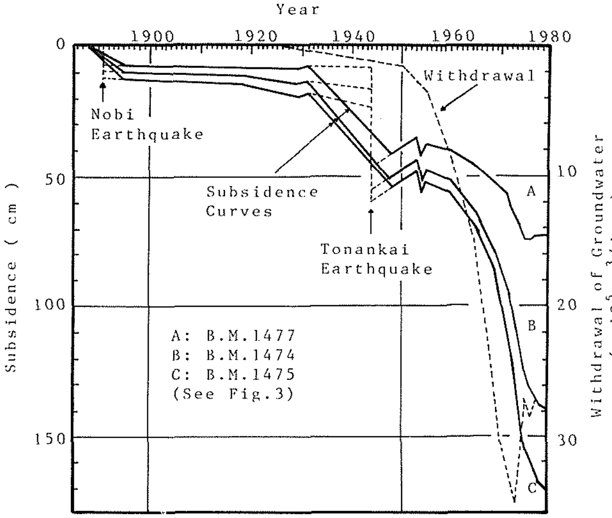

Fig.2 Subsidence of bench marks and withdrawal in the Nobi plain

The major cause of this increasing subsidence was the increasing withdrawal of groundwater from the confined aquifers of the Quaternary deposits (Iida et al, 1976). However the rate of such severe land subsidence is decreasing due to the recent regulation of groundwater use since 1974.

The subsidence of this plain for 21 years from Feb. 1961 to Nov. 1982 is shown in Fig.3. The southern part facing the Ise bay(W1 in Fig.3) settled 157 cm during these 21 years.

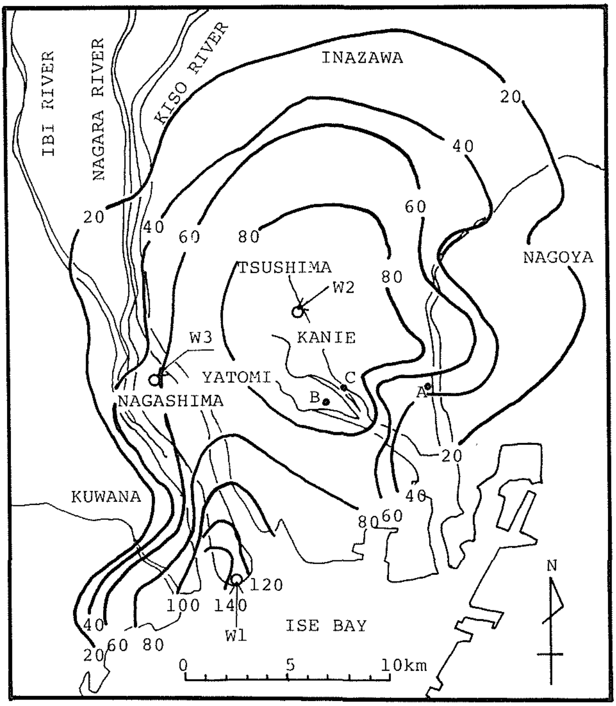

Fig.3 Subsidence for 21 years from Feb.1961 to Nov.1982 in the Nobi plain (cm)

Desirable groundwater level

The authors prepared one-dimensional consolidation model in order to find out desirable groundwater level for preventing land subsidence. This model is able to compute the time-dependent consolidation phenomena of aquitards due to lowering of the piezometric head of groundwater in confined aquifers.

In this analysis groundwater in aquitards was extracted as one-dimensionally vertical flow into aquifers of which piezometric head was dropped down by withdrawal of groundwater. As the displacement of soil is also one-dimensionally vertical one at each computational point, the continuous equation of groundwater flow in a saturated aquitards can be expressed as follows;

$$\frac{\partial}{\partial z}(k_z\frac{\partial h}{\partial z})=-\frac{\partial}{\partial t}(\frac{\partial u}{\partial z}) \qquad (1)$$

where h denotes the piezometric head of groundwater, u denotes the vertical displacement of soil, k_z denotes the vertical permeability and z denotes the vertical co-ordinate.

The equilibrium equation in the saturated soil is also expressed as follows;

$$\frac{\partial}{\partial z}\{E\frac{\partial u}{\partial z}+\gamma_w(h-z)\}+f=0 \qquad (2)$$

where γ_w denotes the unit weight of groundwater, E denotes the modulus of elasticity of laterally confined soil and f denotes the gravity force.

In this analysis, the Tertiary layer was assumed to be impermeable bed rock. Then the boundary condition at the bottom of Quaternary deposits was introduced as follows;

$$k_z\frac{\partial h}{\partial z}=0 \tag{3}$$

$$u=0 \tag{4}$$

The change of piezometric heads of aquifers was known with many observation wells installed in this plain. Accordingly these observed data on the piezometric heads of aquifers were used as known variables in this computation.

As the land subsidence in 1950 was still negligibly small, pore water pressures in aquitards were assumed to be hydrostatic conditions at that time.

Computation of finding out the desirable groundwater level preventing land subsidence was performed for three locations; Matsunaka(W1), Tsushima(W2) and Nagashima(W3), where observation wells W1, W2 and W3 exist as shown in Fig.3.

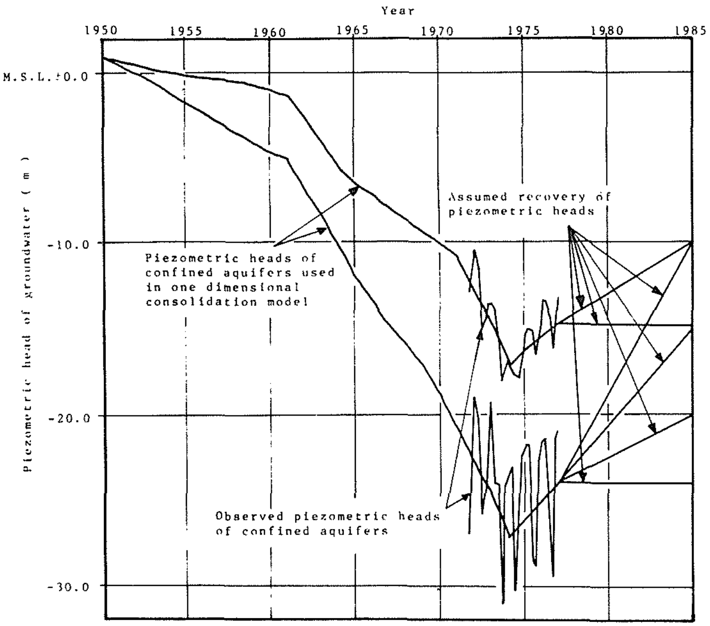

Fig.4 Piezometric heads of confined aquifers in one-dimensional consolidation model

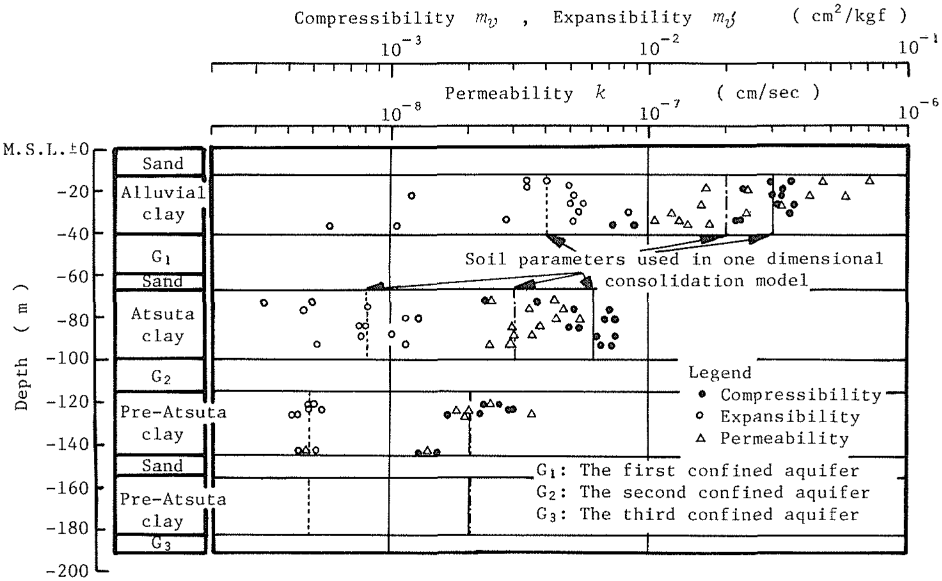

Fig.5 Results of oedometer tests and soil profile

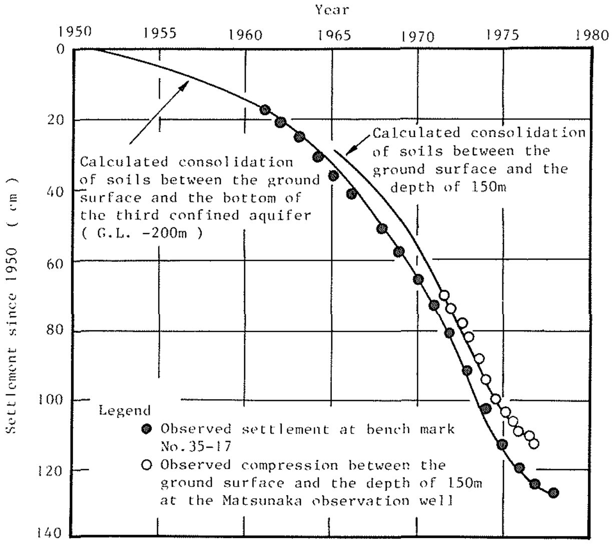

Fig.6 Comparison of calculated settlement with observed one

The piezometric heads of groundwater and displacements of soil layers within the Quaternary deposits were calculated according to the piezometric heads of the 1st, 2nd and 3rd confined aquifers. Nodal points for calculating consolidation phenomena were located at each 1 or 2 meters within aquitards.

Fig.4 shows the observed piezometric heads of confined aquifers at Matsunaka observation well since 1972 and the assumed ones for the calibrating computation of subsidence during 1950 to 1977. Although seasonal fluctuations are seen in the factual piezometric heads, smooth curves on the average were used in this analysis.

For the first trial computation, coefficient of volume compressibility m_v , coefficient of volume expansibility m_v' and coefficient of permeability k were assumed by the mean values of oedometer test results. On the case of Matsunaka, the first assumed coefficients needed not to be changed as shown in Fig.5. But on the other two observation wells, the first estimated ones were modified by the calibrating computation.

The comparison of computed land subsidence with the observed one is shown in Fig.6. Computed pore water pressure could be compared with the observed one of the alluvial clay layer at Matsunaka in 1971 and 1978 as shown in Fig.7. Computed results were proved to have a considerable good agreement with the observed ones from these figures.

Fig.8 shows the predicted rate of land subsidence in 1985 assuming several recovery speeds of groundwater heads as shown in Fig.4. The horizontal axis in Fig.8 is the assumed piezometric head of groundwater in 1985, and vertical axis is the predicted rate of land subsidence in 1985. According to Fig.8, the authors concluded that piezometric heads of G.L.-10 m are desirable groundwater condition to cease land subsidence of the Nobi plain in 1985.

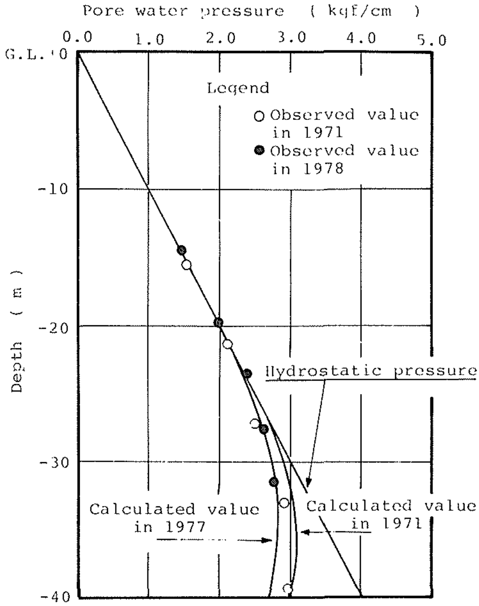

Fig.7 Comparison of calculated pore water pressure with observed one

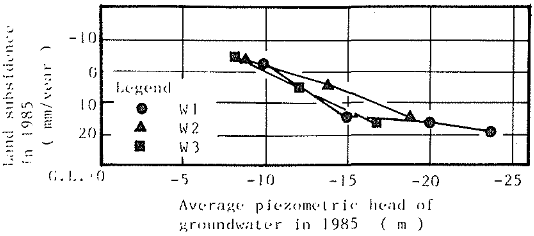

Fig.8 Predicted rate of land subsidence assuming recovered average level of piezometric heads at W1, W2 and W3 (See Fig.3) in 1985

<u>Allowable yield of ground water from the Nobi plain</u>

The groundwater movement in Nobi plain was computed by using the three-dimensional finite element model of this groundwater basin. The future allowable withdrawal of groundwater that will not cause the land subsidence was studied by the aid of this model.

According to Darcy's law and the equation of continuity, the following fundamental equation was applied to the finite element analysis on groundwater flow;

$$\frac{\partial}{\partial x}(k_x\frac{\partial h}{\partial x})+\frac{\partial}{\partial y}(k_y\frac{\partial h}{\partial y})+\frac{\partial}{\partial z}(k_z\frac{\partial h}{\partial z})=s\frac{\partial h}{\partial t}+w \quad (5)$$

where x, y, z denote the Cartesian coordinates, k_x, k_y, k_z denote the coefficient of permeability in x, y, z directions, s denotes the specific storage, h denotes the piezometric head of groundwater and w denotes the average withdrawal of groundwater from a unit volume per unit time.

The Nobi groundwater basin was divided into 2646 tetrahedral finite elements. Fig.9 shows the area of the Nobi groundwater basin model which is 1164 km^2 and covers about 90 percent of the Nobi alluvial plain. Because change of the groundwater head in the unconfined aquifer is small, the groundwater head of unconfined aquifer was assumed to be

$$h = const. \tag{6}$$

The Tertiary strata beneath the Pleistocene were assumed to be impermeable. Then the following boundary condition was introduced at the bottom of this model.

$$k_x \frac{\partial h}{\partial x} l_x + k_y \frac{\partial h}{\partial y} l_y + k_z \frac{\partial h}{\partial z} l_z = 0 \tag{7}$$

where l_x, l_y, l_z denote the direction cosines of outward normal to the boundary surface.

At the boundaries where this model faces to the Ise bay and eastern part of Nagoya city, the following boundary condition was used.

$$q = \alpha (h - h_0) \tag{8}$$

where q is the groundwater discharge through a unit boundary area per unit time, α is the leakage factor, which is determined by the average permeability and distance from the recharging source to this model, and h_0 is the piezometric head of groundwater of adjacent recharging source. Schematic explanation of these boundary conditions are shown in Fig.10.

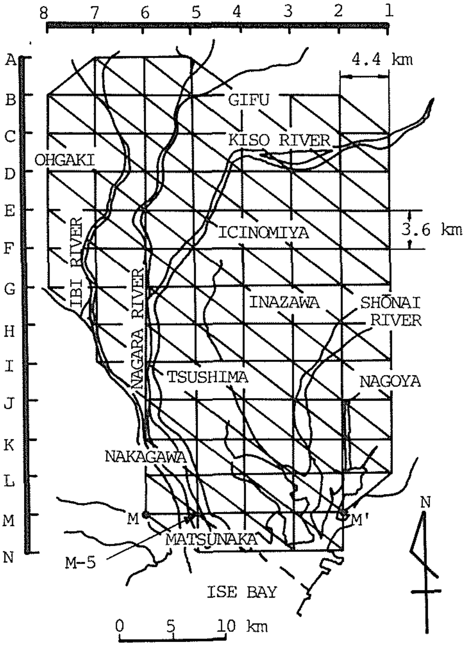

Fig.9 The area of three-dimensional Nobi groundwater basin model

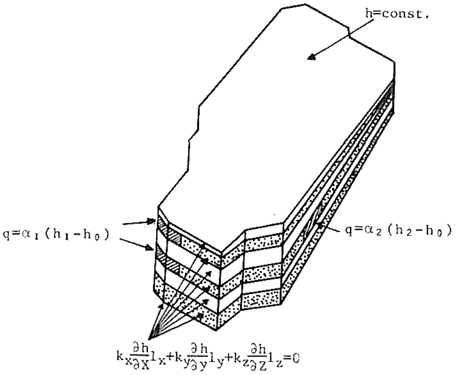

Fig.10 Schematic explanation of boundary conditions

Fig.11 shows the vertical M-M' section of the Nobi groundwater basin model shown in Fig.9. Alluvial and pleistocene strata in the Nobi groundwater basin were modeled as geometrical six layers. The second, the fourth and the sixth layers of the model are corresponding to three main aquifers (G1, G2 and G3), and the first, the third and the fifth layers are compressible layers in the Nobi groundwater basin model.

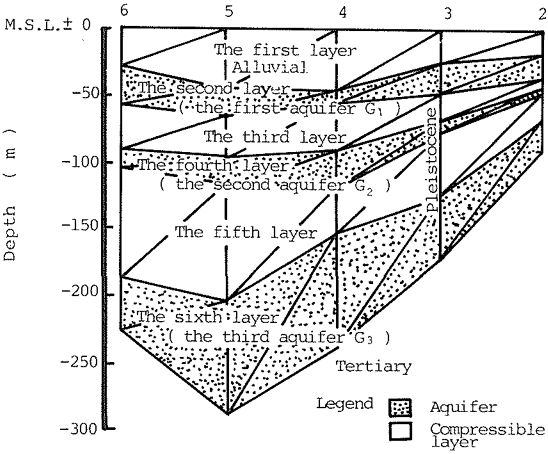

Fig.11 Vertical sectional view (M-M' section in Fig.9)

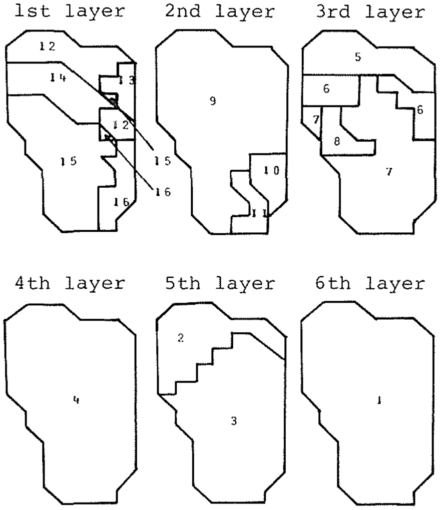

Fig.12 Soil layers of the model

This groundwater basin model was divided to 16 soil layers based on coefficient of permeability and specific storage as shown in Fig.12.

The coefficient of permeability and specific storage of this model were determined based on various soil test data and many pumping test data. These soil parameters were used as the soil parameters of the first trial computation from 1961 to 1977. Comparing the computed piezometric heads of groundwater with the observed ones, these soil parameters were modified by the trial and error method until computed piezometric heads of groundwater became similar to the ones of observation well.

Result of calibrating computation in Fig.13 shows the computed piezometric heads of groundwater at a nodal point near Matsunaka observation well(M-5 shown in Fig.9). The observed piezometric heads of groundwater are shown in Fig.13. The computed piezometric heads are similar to the average trends of observed ones. Allowable yield that would recover the piezometric heads of confined aquifers to desirable groundwater level was studied with this model and the work described in the preceding chapter. The computations were performed for five cases of the future withdrawal, assuming as follows;

(a) Since 1978 the withdrawal of groundwater would be kept at the yield of 1977.

(b) The yearly withdrawal of groundwater would be reduced by 20 percent of the yield of 1977 since 1978.

(c) Following the condition (b), the yearly withdrawal of groundwater would be reduced by 40 percent of the yield of 1977 since 1979.

(d) Following the condition (c), the yearly withdrawal of groundwater would be reduced by 60 percent of the yield of 1977 since 1980.

(e) Following the condition (d), the yearly withdrawal of groundwater would be reduced by 80 percent of the yield of 1977 since 1981.

The results of the computation at the nodal point M-5 (see Fig.9) are shown in the period of prediction of Fig.13. From this figure it is concluded that the future withdrawal must be decreased to about half of the yield of 1977.

Prediction of land subsidence

Prediction of land subsidence by lowering of piezometric heads due to the withdrawal of groundwater was carried out by using the three-dimensional and one-dimensional finite element models. The three-dimensional model was used to estimate the relation between the withdrawal of groundwater and the piezometric heads of confined aquifers. The one-dimensional model was used to calculate the land subsidence in relation to change of the piezometric heads in confined aquifers estimated by the three-dimensional model (Ueshita and Sato, 1981). Fig.14 shows the predicted results at Matsunaka observation well W1 (see Fig.3) and the observed data obtained at bench mark No.35-16 installed near this observation well. Solid lines in this

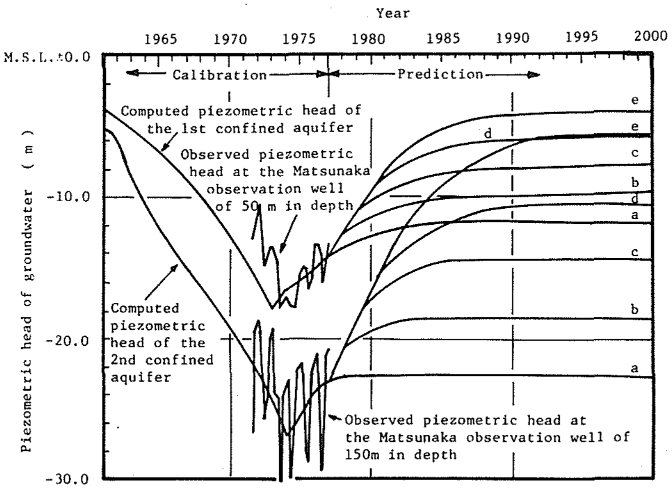

Fig.13 Computed piezometric heads of groundwater at M-5 point in Fig.9 and observed data

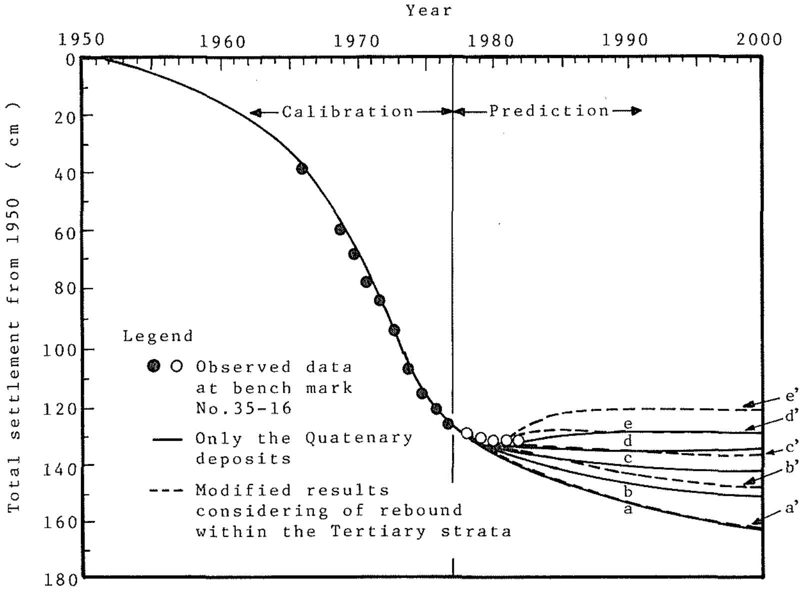

Fig.14 Predicted and observed land subsidence at bench mark No.35-16 near Matsunaka observation well of the Nobi plain

figure are computed results neglecting the movement at the base of model. From these results it was shown that the recent withdrawal of groundwater in this area decrease corresponding to the withdrawal condition between (d) and (e).

Fig.15 shows observed settlement of bench mark No.35-16 and observed consolidation of layers at the Matsunaka observation well. From these observed data, it is cleared that rebound is occurring in the layers deeper than G.L.-150 m. The recovery of groundwater level in these deep layers makes the rebound of these layers as a result of the regulation of withdrawal of groundwater in this plain since 1974.

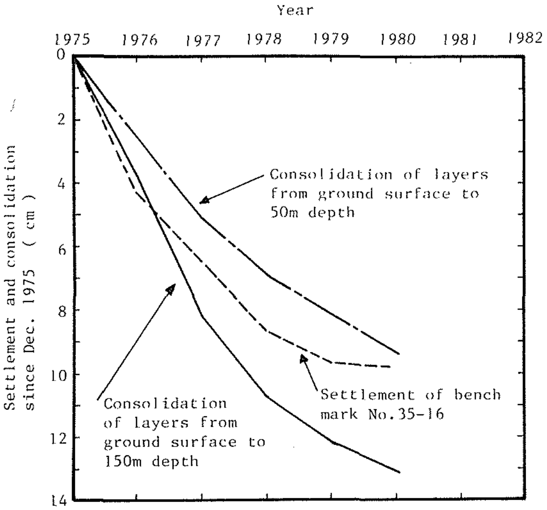

Fig.15 Settlement of bench mark and consolidation of layers at Matsunaka observation well

This rebound phenomenon at the Tertiary surface can be seen also by other observation wells. Fig.16 shows the relation between the rebound and the recovered head of groundwater in the third confined aquifer. According to this figure, about 7 mm of rebound at the Tertiary surface was known per 1 m recovered head of groundwater in the third confined aquifer.

The former predicted results were modified as shown in dashed lines of Fig.14 by use of the relation between rebound of the Tertiary strata and the recovered groundwater head of the third confined aquifer. From these modified results, the recent withdrawal of groundwater proved to decrease corresponding to the yield plan (c').

Fig.17 shows the investigated withdrawal in the coastal area of the Nobi plain. From this figure it is assured that the recent withdrawal of groundwater is decreasing the yield plan (c). Then the authors concluded that the precise prediction of land subsidence should take account of the movement at the base of model.

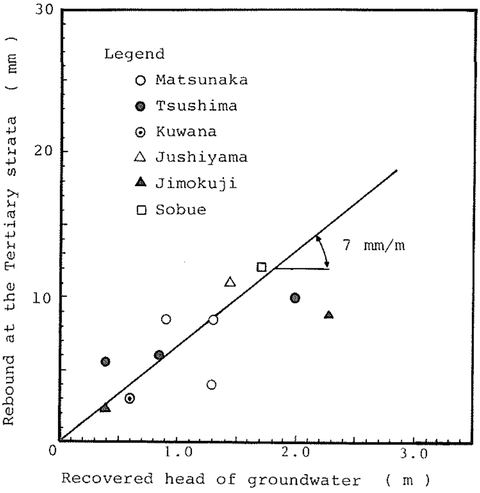

Fig.16 Rebound at the Tertiary strata and recovered head of groundwater

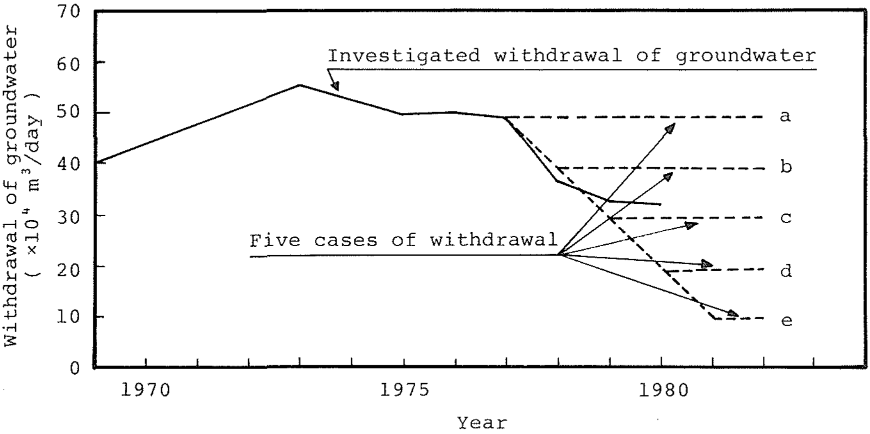

Fig.17 Recent withdrawal of groundwater at coastal area of the Nobi plain

Acknowledgement

The authors would like to express their hearty gratitude to the Tokai Three-Prefecture Investigation Committee on Land Subsidence and to the organs and persons concerned.

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HYDROGEODEFORMATION FIELD IN STUDYING PROCESSES OF LAND SUBSIDENCE

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Abstract

An extensive development of natural resources, fluid mineral deposits in particular, results in sharp and sometimes irreversible environmental consequences. Such unfavourable effects also incorporate processes of land subsidence frequently encountered on the globe and attributed to a mass extraction of ground water, oil and gas followed by the pressure drop in the subsurface and rock consolidation. Presently, re-levelling that embraces large time intervals and provides registration of changes in the position of the ground surface amounting to several millimeters per year, is one of the basic observation methods to be applied toward the study of this process. Studying hydrogeodeformation field that represents a new specific type of a geophysical field, enables the specialists to determine the mechanisms of subsidence processes as a result of a change of the stressed state in the collector used and in its surroundings, to register the trends in the development of deformation processes, and to predict the probability of disastrous land subsidences.

Discussion

Alongside favourable effects, an extensive economic development in vast regions is accompanied by some negative processes that result in the failure of large land areas, land-slide formation, water level rise at urban and agricultural territories, etc.

Among such consequences that produce considerable economic losses in many countries, processes of land subsidence caused by a combined impact of mining and oil industries, extensive ground-water withdrawal, natural and technogenic karst, etc. can be distinguished.

Being low-amplitude processes at the initial stages of development, they acquire considerable scales at the mature stage with a cauldron embracing areas of tens and sometimes hundreds of square kilometers.

Hence, a time identification of trends in the development of the internal processes which can lead to irreversible land subsidences is considered to be the most important practical problem in modern engineering geology and hydrogeology.

A universal occurrence of a stress field accounting for a distribution of compression and extension areas in a rock mass was established in the course of engineering activity, mining, production of hard, liquid and gaseous mineral deposits.

Recently, in analyzing various aspects of interrelations between the earth's stress field and geologic, geophysical, hydrogeological information, one can assume the existence of definite regularities attributed to genetic requisites of such relations. Determination of these regularities is no doubt of importance for the rational use of the subsurface, as well as for correct and time management of the processes that originate or are activated under a man's geoenvironmental impact.

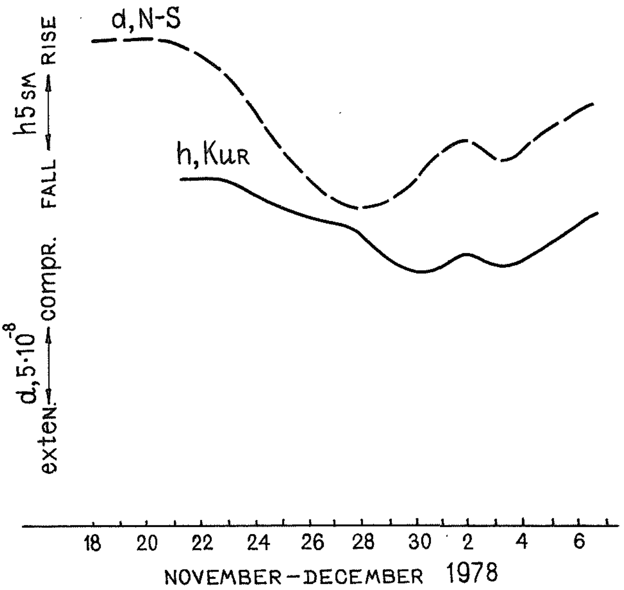

Fig. 1—Water table variations in a well (h) and earth surface deformations (d), (by Monakhov, Khantaev, Saprygin, 1979)

A new unknown before phenomenon of global transient pulsating variations in hydrogeosphere attributed to an ability of the latter to respond to a change of the stressed state in the lithosphere (Vartanyan, Kulikov, 1982, 1983) was established in the USSR as a result of a series of longterm studies concerning the regularities of changes in the ground-water regime under various natural environments and technogenic impact. This phenomenon was registered as a discovery in the state list of the USSR, and in essence it describes rather a unique hydrogeodeformation (HGD) field.

The main regularities in the development of the HGD-field under natural and disturbed conditions were studied using specially developed methods of investigating the stressed state in rock masses. This enabled determination of the main features of the HGD-field, as well as an assessment of the potentials for its use toward the solution of some applied problems. Constant subsurface redistribution of fluid components in the lithosphere due to a change of natural (endo-, exo- and technogenic) stresses in its solid section, as well as due to formation and decay of numerous thermodynamic, physical-chemical anomalies that control the areas of short-lived (hours, days, months) microdeformation processes was demonstrated.

Thus, the HGD-field is a resultant of a simultaneous effect of numerous varying in force and trend processes of a matter deformation in a total rock volume.

Here, as implied by the specific features of the HGD-field development, not all of the deformation processes are to result in the formation of disruptive or plicative dislocations. The latter evidently are the final forms of a long-term oriented development in large lithospheric sections, as well as particular rock masses in the zones of intensive technogenic processes. In this case, the short-lived strain structures composing a spatial fabrique of the HGD-field reflect a pulsating in real time regime of a change of the physical state in a study geologic section.

Recently, the relation between some hydrogeological parameters (heads,

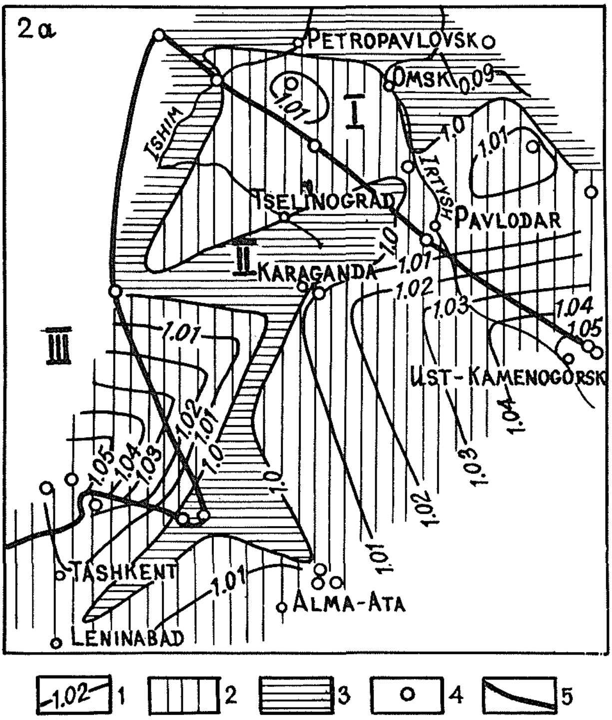

Fig. 2a—Map of hydrogeodeformation field in West Siberian-Middle Asian segment (15 January 1971). 1 - isolines of equal rock deformations (conv. un.); 2 - relative extension zones; 3 - relative compression zones; 4 - wells; 5 - structural zone boundaries: I - West Siberian plate (south-eastern section), II - Tyanshan - Kazakh mobile region, III - Turan plate.

water temperature, etc.) and the degree of rock deformation has been determined.

Here, a large series of experiments performed show absolute coincidence of the "compression - extension" curve derived using demographs toward any particular region and the curves showing the depth of water table in the wells of this region (Fig. 1).

While regarding this relation, for example, an approximate assessment of rock deformations on the amplitude of ground-water table variations becomes possible. In particular, using the data on measurements performed at Sakhalin-Kurily test ground it can be assumed that each centimeter of water level drawdown here corresponds to a value of relative extension $(d\ .9.10^{-9})$.

Existance of the HGD-field and the short-lived strain structures can be comprehended judging from a common proposition that sounds as follows: in a certain filtration area governed by the law of a fracture porosity in a mass, processes of heat and mass transfer will occur, according to the nature of a spatial distribution of the adequate physical-chemical and thermodynamic potentials, and will be recorded using values of the head, concentration, temperature, gas elasticity, etc.

Under constant conditions at the external boundaries the character of the fluid distribution in a collector is governed by the laws of potential heat- and mass transfer, and appears to be stable with time. Given a dis-

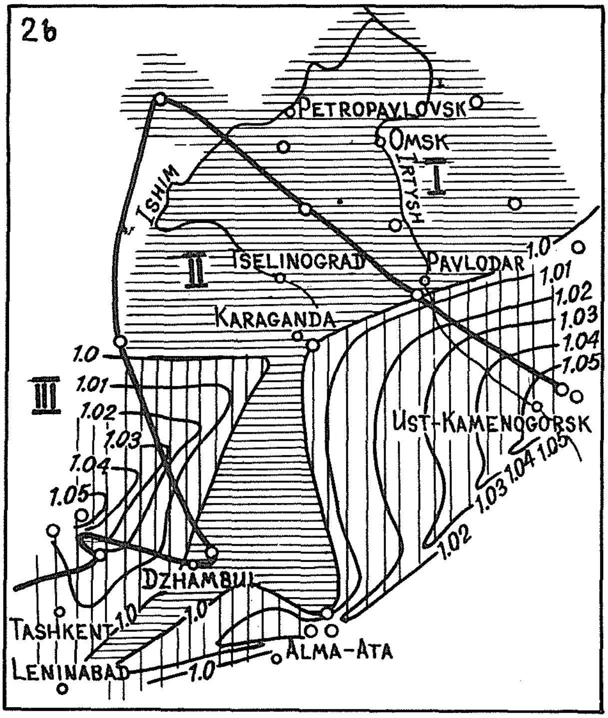

Fig. 2b--Map of hydrogeodeformation field in West Siberian-Middle Asian segment (13 January 1971). 1 - isolines of equal rock deformations (conv. un.); 2 - relative extension zones; 3 - relative compression zones; 4 - wells; 5 - structural zone boundaries; I - West Siberian plate (south-eastern section), II - Tyanshan-Kazakh mobile region, III - Turan plate.

turbed initial state of a confined water system (storage capacity), anomalies presented by the changes in filtration flow structure (its hydrodynamic, concentration, thermal and other constituents) occur in conformity with the nature, power and point of application of a disturbing impulse.

Hence, a reconstruction of the collector capacity can be regarded as the HGD-field model that objectively reflects a stressed state in the subsurface during various periods of time.

Such reconstructions may be observed only provided that mechanical stresses and relative strains in a mass are developed much faster as compared to fluid flow processes in a collector. Otherwise, a relaxation of geodynamic anomalies attributed to filtration processes would take place.

To accomplish an operative observation of the process development an automatized information system "Moire" was constructed that provides compilation of the HGD-field maps and plotting the deformation curves which describe the rates and trends in the short-lived structure development.

Studying the HGD-field in large regions of the USSR enabled establishment of the specific features in the origin and development of the short-lived strain structures.

It has been defined that a standard HGD-field either in mountain-folded regions, or within the limits of platforms and shields possesses an irregular structure, i.e. extension zones in the form of isolated box structures appear and degrade among the sites of weak compression during short-time periods (Fig. 2a). Such morphological field variations have a flickering nature. A rather distinct regularity is noted: tensile strain developed at a particular area relaxes, on reaching a definite limit, and is replaced by a relative compression, and v.v.

Also an existence of an abnormal HGD-field is registered, i.c. particular isometric spots of the strain structures are increased in size, elongate via their merging and forming linear extended structures (Fig. 2b).

The entire HGD-field becomes structure-oriented, ie. the elongated extension zones alternate or intersect the extended compression areas.

Such transformations of the HGD-field evidence the occurrence of fast pulsating changes in physical parameters of enormous undisturbed rock masses.

It is apparent that in the regions subject to an active technogenic impact the HGD-field will acquire specific features indicating the course of the processes of rock consolidation, strata pressure drop, rock mass deformation and sliding into a cauldron.

High sensitivity of the HGD-field to microstrain alterations in rock masses may be the most important indicator at the initial stages of cauldron formation. Therefore, in this case, quite natural is the appearance of a specific instrument to be used in land subsidence monitoring, time developmen of the solutions that provide optimization in withdrawal of ground water and other fluids, as well as consideration of the problems concerning the geoenvironmental protection against unfavourable effects.

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ASSESSMENT OF THE TRANSIENT NATURE OF SUBSIDENCE

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Abstract

To meet the demand for agriculture land and housing in populated delta areas wetland is reclaimed into polders, i.e. lowlands with precise water-regime control. This will change the geo-hydrological system and may cause land subsidence in the environment. Increased withdrawal from groundwater storage causes piezometric drawdown and may result in subsidence. Oil and gas production by compaction drive induces similar effects. The reservoir shrinks creating subsidence at the landsurface.

Simulation models are applied to evaluate and predict these environmental effects. The main problem is the schematisation of the actual complex geo-hydrological and geotechnical system. Since acquired date are incomplete, sophisticated models cannot be tuned properly. One is apt to apply simple models, but most of them are not reliable to predict subsidence behaviour. To release this quandary a new approach is suggested: the method of the transient leakage factor. This approach yields simple models yet covering the essential phenomena properly. The present contribution deals with the implementation of the approach with the emphasis on time-variant and three dimensional effects.

Land subsidence modelling

The general assessment of problems related to landsurface subsidence due to fluid withdrawal from aquifers and reservoirs is to be performed by the following activities:

- recognition of the geo-hydrological system;
- selection and development of simulation models;
- field monitoring and verification.

In this paper the modelling of subsidence is considered. For the numerical modelling of subsidence various levels of approach can be distinguished (viz. Saxena,1978; Helm,1982; Coats,1982):

- black box model. This aproach is based on a purely empirical concept. Observed land-subsidence is related to total production from aquifers and reservoirs (global mass-balance). Forecasting subsidence by extrapolation of black box data is unreliable.

- uncoupled model. This model represents the conventional approach. It consists of a regional model, that generates the reservoir compaction, the actual pressure and discharges, and a consolidation model, that simulates the effects of slowly-draining intermediate clay layers also contributing to the observed land subsidence. The calculated pressure in the reservoirs represent the boundary conditions for the consolidation process in these clay layers. The flow field and the consolidation process are uncoupled.

- semi-coupled model. This approach is similar to the uncoupled model, but processes in the aquifers and reservoirs are coupled to the slowly draining intermediate clay layers. The drainage from consolidating layers is included in the flow system. The complete system can be solved on the basis of approximate analytical solution techniques.

- fully-coupled model. This is physically the most complete approach. In principle three-dimensional processes are considered including horizontal deformations. This approach demands many parameters and is relatively costly. It is a valuable tool to study involved phenomena and to evaluate some simplified models. For many problems, particularly for large regions this level of sophistication is not recommended, and often not necessary. In principle one can state that the level of approach to be chosen depends on the quality of available data. Essential input for the subsidence forecast is knowledge about the production strategy.

Transient nature of subsiding systems

Geo-hydrological systems revealing landsurface subsiding caused by fluid withdrawal are conceptually well classified as reservoir- or aquitard-drainage models, in which less pervious intermediate layers (aquitards) serve as hydraulic separators. If only vertical deformation is considered, two components can be distinguished in the subsidence, to wit: vertical deformation in the reservoir, called compaction, and vertical deformation in the aquitards, called consolidation. The time dependent behaviour is formulated by means of storativity of the aquifer, while the contribution from aquitards is expressed as leakage, commonly described by a constant leakage factor and proportional to the reservoir pressure.

This concept has been suggested already by Jacob (1940) and it has been applied widely since. However, for time-dependent flow conditions the contribution of aquitards cannot be properly described by a constant leakage factor. Only in special situations where the compaction is dominating the consolidation (van der Knaap, 1967) Jacob's concept applies.

To show the difference between consolidating and compacting systems the following five models are considered; each one simulates the piezometric response of a leaky aquifer to a time-variant plane-symmetrical boundary condition: $H(x,t) = H_0\cos(\omega t)$.

Model I: a rigid aquifer and a rigid aquitard with a constant leakage factor. The response in the aquifer becomes:

$$H(x,t) = H_0 \exp(-x/\lambda)\cos(\omega t) \qquad (1)$$

Model II: a deformable aquifer (elastic storage) and an impervious upper layer. The response in the aquifer becomes (Verruijt, 1982):

$$H(x,t) = H_0 \exp(-x/\lambda_\omega)\cos(\omega t - x/\lambda_\omega); \quad \lambda_\omega = \sqrt{2c/\omega} \qquad (2)$$

in which c represents the aquifer compaction coefficient, related to the aquifer storativity S by: $S = KD/c$ (KD is the aquifer transmissivity).

Model III: a rigid aquifer and a deformable aquitard. In the aquitard the vertical consolidation process is formulated according to Terzaghi by the consolidation coefficient c' (see Verruijt, 1982a). The response becomes:

$$H(x,t) = H_0 \exp(-x/\lambda_\omega)\cos(\omega t - 0.41\ x/\lambda_\omega); \quad \lambda_\omega = 0.91\lambda/\sqrt{\delta}; \quad \delta = d\sqrt{\omega/2c'} \qquad (3)$$

Model IV: a deformable aquifer and a rigid permeable aquitard. The leakage is directly related to the changes in the aquifer piezometric head H ; no time delay due to processes in the aquitard are considered. This model is common for the pumping test evaluation (Hantush, 1964). The response is:

$$H(x,t) = H_0 \exp(-x/\lambda_\omega)\cos(\omega t - \alpha x/\lambda_\omega); \; \lambda_\omega = \lambda \frac{\sqrt{1+\alpha^2}}{\sqrt{1+\beta^2}}; \; \begin{array}{l} \beta = \omega\lambda^2/c; \\ \alpha = \tan(\tfrac{1}{2}\mathrm{atan}\beta) \end{array} \tag{4}$$

Model V: a deformable aquifer and a deformable aquitard. The response in the aquifer becomes (Barends, 1983):

$$H(x,t) = H_0 \exp(-x/\lambda_\omega)\cos(\omega t - \alpha x/\lambda_\omega); \; \lambda_\omega = \lambda \frac{\sqrt{1+\alpha^2}}{\sqrt{\delta\sqrt{1+\varepsilon^2}}}; \; \begin{array}{l} \varepsilon = 1 + \beta/\delta \\ \alpha = \tan(\tfrac{1}{2}\mathrm{atan}\varepsilon) \end{array} \tag{5}$$

The corresponding pore-pressure in the aquitard is:

$$h(x,z,t) = H_0 \exp(-x/\lambda_\omega - z\sqrt{\omega/2c'})\cos(\omega t - \alpha x/\lambda_\omega - z\sqrt{\omega/2c'}) \tag{6}$$

and the total vertical (cyclic) subsidence becomes:

$$s(x,t) = (KD/c)H(x,t) + \int_0^d (K'/c')h(x,z,t)\,dz \tag{7}$$

The difference between the various simulation models becomes apparent when an observed leaky aquifer behaviour is elaborated. The considered observation concerns a tidal response in a coastal leaky aquifer (see Figure 1). The graphs show the correlation of the piezometric head at two positions, one at the coastline (point A) and one 50m inland (point B). Several tides have been recorded. The plotted data can be visualized as tilted ellipses, which incorporate amplitude decay and delay. For the previously mentioned models the data gives: $\lambda_\omega = 120$m and $\alpha = 0.58$. Next, characteristic parameters for the geo-hydrological system can be determined, the leakage factor and the storativity. However, the calibrated values depend strongly on the simulation model chosen. The following table shows the results.

	model			result	
	reservoir	reservoir -	environment	λ/λ_ω	S/KD
	compaction	permeable	consolidation		
I	no	yes	no	1.00	-
II	yes	no	no	-	6.25
III	no	yes	yes	3.45	-
IV	yes	yes	no	1.25	3.55
V	yes	yes	yes	3.85	1.55

A prove that a leaky aquifer indeed responds to tidal fluctuations is presented in Figure 2, showing very accurate subsidence measurements in the gasfields in the north of the Netherlands. Since the aquitard might play a major role in the subsidence process, it is important to select the model that simulates the land subsidence due to fluid withdrawal in a physically consistent way. Models III, IV or V are suitable for subsidence; model III and V for consolidation of the environment (aquitards); model IV and V for reservoir compaction (aquifer). In model V the flow process in the aquifer and the consolidation process in the aquitard are covered in a physically consistant way. A simple method is developed to overcome the more complex formulation of this model: the method of the transient leakage factor (see Barends, 1982). This approach is classified as a semi-coupled model. It is applicable for regional subsidence problems.

Another striking example of the time-variant behavior of subsiding geohydrological systems can be shown on the basis of the complete solution of

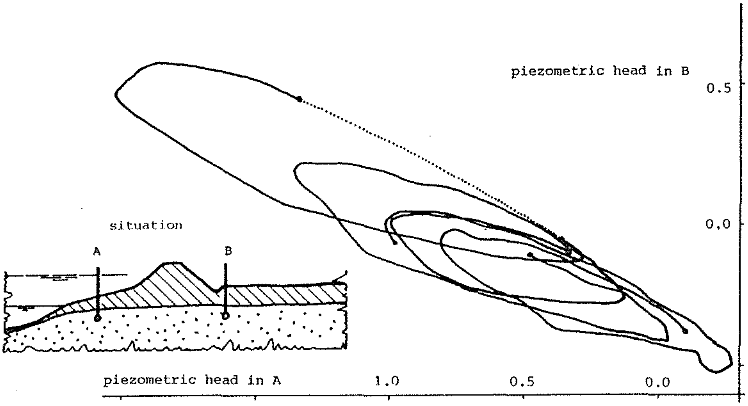


FIG. 1 Decay and delay effects in a reservoir response (leaky aquifer)

the drawdown in a leaky aquifer system due to a steady well operating from time t=0. The solution is (model IV; Hantush,1964):

$$H(r,t) - H_0 = (Q/2\pi KD)\ W(\alpha,\rho);\ \ \alpha = r/2\sqrt{ct};\ \ \rho = r/\lambda \tag{8}$$

where c is the aquifer compaction coefficient, λ is the leakage factor and W is a well-function, defined according to:

$$W(\alpha,\rho) = \int_{\alpha}^{\infty} \exp(-u^2 - (\rho/2u)^2)/u\ du \tag{9}$$

If slowly-draining layers contribute to the process, model V applies. The analytical solution exists for a thick aquitard, according to:

$$H(r,t) - H_0 = (Q/2\pi KD)\ B(\alpha,\beta);\ \ \alpha = r/2\sqrt{ct};\ \beta = \rho^2\delta;\ \delta = d/2\sqrt{c't} \tag{10}$$

where the function B is defined according to:

$$B(\alpha,\beta) = \tfrac{1}{2}\int_{\alpha}^{\infty} \exp(-u)\,\mathrm{erfc}(\beta/\sqrt{u(u-\alpha)})/u\ du \tag{11}$$

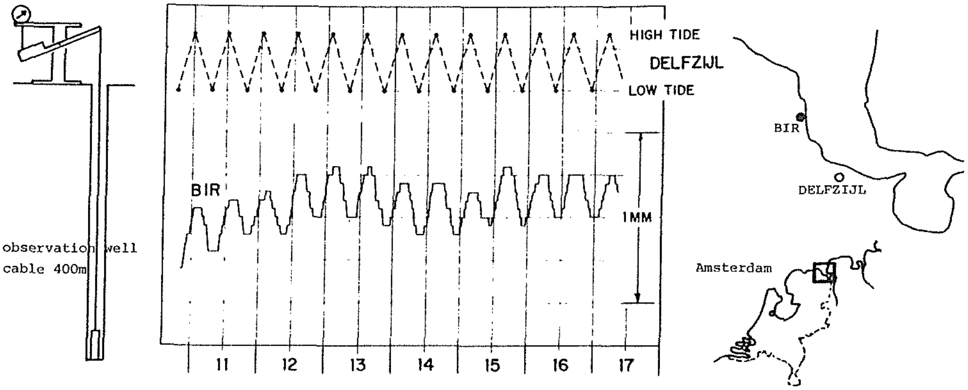


FIG. 2 The measured response of a coastal leaky aquifer to tides.

The approximate solution according to the method of the transient leakage factor yields a similar solution, but now the function B simplifies to:

$$B(\alpha,\beta) = K_o(r/\lambda_t);\ \lambda_t = \lambda / \sqrt{(\lambda^2/2ct)} + (d/\sqrt{2}c't) \qquad (12)$$

This expression is identical to the standard solution for a semi-confined aquifer (see Verruijt, 1982) but with a time-variant leakage factor. The solution shows similarity to a formula valid for semi-phreatic systems (is then a Boulton's drainage factor).

In Figure 3 the various formulas are presented in a dimensionless form. The shape of the reservoir pressure distribution is in fair agreement for all solutions. However, the essential difference is the time dependency, since the ordinate for model IV is: $r/\sqrt{ct}$, and for model V: $r/\sqrt{KD\sqrt{c't}/K'}$. This implies that the process of compaction is developing proportional to $\sqrt{t}$, whereas the consolidation proceeds accoring to $\sqrt[4]{t}$. Consolidation in the intermediate slowly-draining layers will proceed at lower speed. It is significant for the evaluation of the final subsidence.

Inhomogeneities in the considered system can be accounted for by adopting double sources along the interfaces of inhomogeneity with a strength to be determined from continuity requirements. The field effect of these sources is formulated by equations (10) and (12).

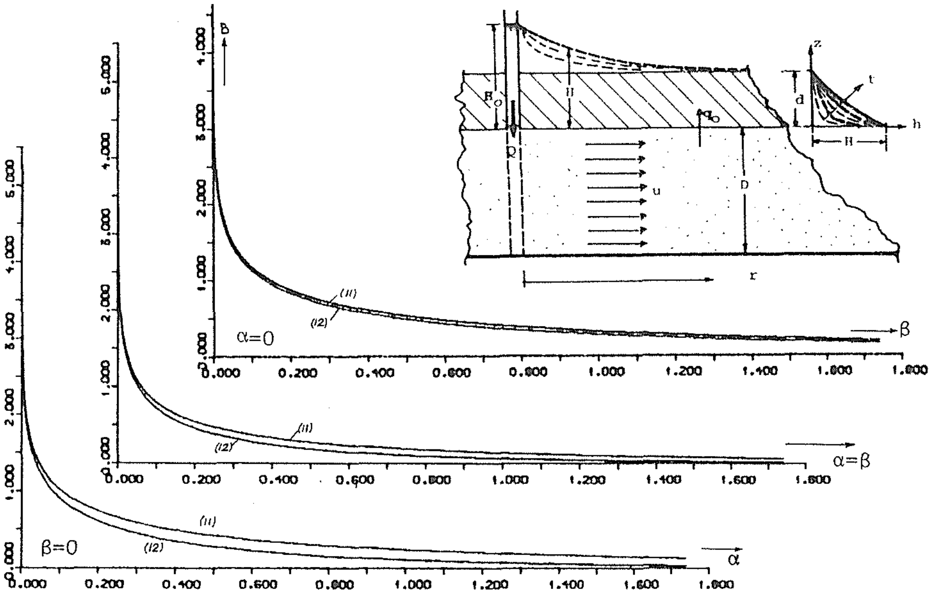


FIG. 3 Comparison of various models: a steady well in a leaky aquifer

Implementation of the transient nature of subsidence

In many geohydrological systems subsidence is due to consolidation in the intermediate and adjacent slowly-draining layers. In loose unconsolidated reservoirs compaction forms a major cause to subsidence (Abou-Sayed,1982). Because the consolidation and deformation response in the environment are due to the reservoir behaviour (Geertsma,1963;Gambolati,1973), the environmental behaviour must be considered, particularly with respect to final subsidence prediction. Knowledge about the deformation and consolidation behaviour is essential for the evaluation of subsidence.

To formulate subsidence the essential phenomena involved must be qualified and the dominating process must be selected. In the previous section a simple formula has been presented, that covers reservoir compaction and environmental consolidation. A formula also valid for the leakage phase is available. The transient leakage factor becomes (Barends,1983):

$$\lambda_t = \lambda / \sqrt{(\lambda^2/2ct) + (1/\sqrt{2\tau})(1+\exp(-\sqrt{2/\tau}))/(1-\exp(-\sqrt{2/\tau}))}; \; \tau = c't/d^2 \quad (13)$$

The time regimes of the various processes involved can be evaluated on the base of this expression. Two critical values appear: $t_1 = \lambda^2/c$ and $t_2 = d^2/c'$. For a sudden disturbance the following is found:

period	dominating process
$t < \mathrm{Min}(t_1, t_2)$	compaction and consolidation
$\mathrm{Min}(t_1, t_2) < t < \mathrm{Max}(t_1, t_2)$	compaction or consolidation
$\mathrm{Max}(t_1, t_2) < t$	constant leakage

Obviously the ratio: $N = t_2/t_1$, is a key to distinguish between a consolidating system ($N >> 1$) and a compacting system ($N << 1$). Close inspection of the number reveals, that it represents the ratio of vertical strain by consolidation and vertical strain by compaction.

The number N is a simple instrument to characterise subsiding systems, using approximate values of a limited number of basic material properties: layer-thickness, permeability and soil compressibility. It is essential to insert however parameter values relevant to the process conditions. Granular deposits reveal a typical non-linear behaviour, related to the type, origine and in-situ state and to the geological history (the maximum burrial stress). In weak rock pore collaps may occur under specific stress conditions (Figure 4) A adequate material investigation is required.

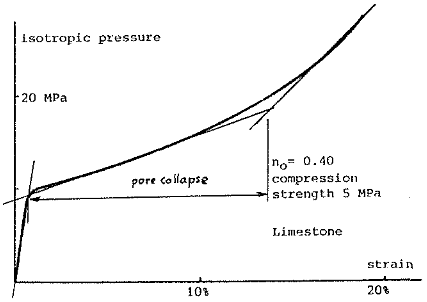


FIG. 4

Consolidation of adjacent layers is strongly effected by sand layers or lences, which accelerate the drainage drastically. A laboratory test will probably show a longer draining period than observed in the field (van der Knaap,1963). Three-dimensional constraints may contribute to this, and the behaviour of fluid-gas mixtures requires more attention (Teunissen, 1982). Recognition of the stratification of the environment by boring and tests on samples provides sufficient information to determine a field response consolidation factor (Vreeken & van Duyn,1983). It may differ some orders of magnitude from values determined in the laboratory.

Application for a regional subsidence problem

In the Netherlands the lowlands are situated in an area with a typical geo-hydrological stratification. The toplayer consists of continuous semi-pervious soil deposits (holocene), and underneath a thich permeable sand deposit (pleistocene). In this sandy aquifer some large clay lences occur.

In a large fresh-water lake, the IJselmeer, a new large polder is to be reclaimed. This polder, the so-called Markerwaard is situated close to the province of North-Holland, which contains many polders with a precise water control (Hannink & Talsma, 1984). The new polder will cause changes in the existing geo-hydrological equilibrium. A new equilibrium is only attained at the completion of the consolidation of the semi-pervious top layer. The following questions arise. How large is the final induced subsidence in the environment? How fast is this attained? For this purpose a semi-coupled aquitard-drainage model is developed applying the method of the transient leakage factor taking into account the inhomogeneity due to the presence of large glacial clay lences (Figure 5).

Two stages are to be considered: the undisturbed initial stage and the transient stage. The initial stage represents the steady-state situation before reclamation. A constant leakage factor applies, thus model I. The transient stage concerns the time-variant situation due to lowering of the water table inside the new polder, and model III or V applies. A boundary lake is preserved between the new polder and the North-Holland border. An axially symmetric segment is considered. The field equation describing the initial stage is (Verruijt, 1982):

$$(1/r)d(rdH_1/dr)/dr = (H_1-H_0)/\lambda^2 \tag{14}$$

where λ is the leakage factor: $\lambda = \sqrt{KDC}$, $C=d/K'$ is the aquitard resistance, K' is the aquitard permeability and d the aquitard thickness. λ is sectionally constant. Applying the method of linking flow fields yields the following solution:

$$r < L \qquad H_1(r) = H_m + (1-f_1)(H_p-H_m)I_0[r/\lambda_m]/I_0[L/\lambda_m] \tag{15a}$$

$$r > L \qquad H_1(r) = H_p + \quad f_1\ (H_m-H_p)K_0[r/\lambda_p]/K_0[L/\lambda_p] \tag{15b}$$

in which the factor f_1 embodies the inhomogeneity, according to:

$$f_1 = 1/(1+ (KD/\lambda)_p(\lambda/KD)_m(K_1[L/\lambda_p]/K_0[L/\lambda_p])(I_0[L/\lambda_m]/I_1[L/\lambda_m])) \tag{16}$$

This factor is a smooth function tending to 0.5 for large values of L/λ .

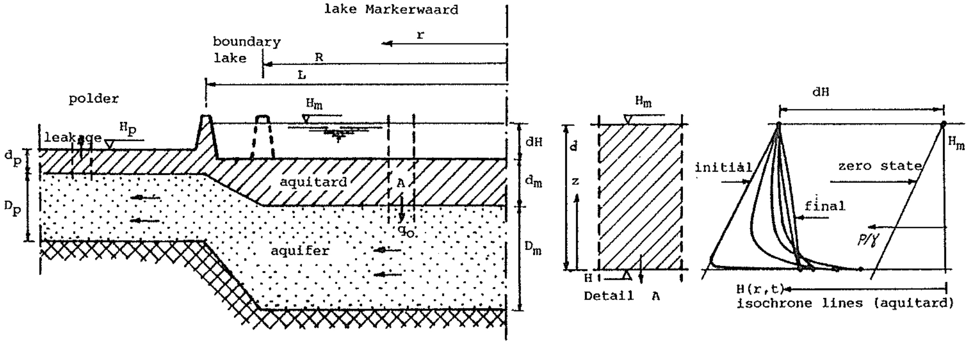


FIG. 5 The geometrical situation (elevation plane).

The transient stage includes the consolidation of the top layer:

$$\partial^2 h/\partial z^2 = \partial(h-\sigma/\gamma)/c'\partial t;\ \sigma:\text{ total stress};\ h = H_2, z \to 0;\ h = dH, z \to d \qquad (17)$$

Presume an intitial stage at rest for this problem. Laplace transform and solution of the consolidation process under transformed transient boundary conditions yields an expression for the flux at the interface:

$$\overline{q_o} = K'\sqrt{s/c'}\ \coth(d\sqrt{s/c'})\ (\overline{H}_2-\overline{dH});\quad s:\text{ transform coordinate} \qquad (18)$$

The overbar denotes a Laplace transform. This flux represents the actual leakage. It is not proportional to the aquifer potential: thus, a constant leakance does not apply. The flow in the aquifer has to be considered with the actual leakage. The field equation is (Verruijt, 1982):

$$(1/r)\partial(r\partial H_2/\partial r)/\partial r = q_o/KD + \partial H_2/cKD\partial t \qquad (19)$$

The field is sectionally homogeneous with respect to the transmissibility and the outer potential. The corresponding leakage at the interface is:

$$r < R \qquad \overline{q_o} = K'\sqrt{s/c'}\ \coth(d\sqrt{s/c'})\ (\overline{H_2}-dH/s) \qquad (20a)$$

$$r > R \qquad \overline{q_o} = K'\sqrt{s/c'}\ \coth(d\sqrt{s/c'})\ \overline{H_2} \qquad (20b)$$

Substitution into the field equation yields a set of equations, which can be solved by the linking flow field technique. The last step is to obtain the inverse transform. The standard solution is quite cumbersome, but the type of problem permits to apply the method of the transient leakage factor. The final solution becomes:

$$r < R \qquad H_2(r,t) = adH(1+(f_2-1)I_o[r/\lambda_{tm}]/I_o[R/\lambda_{tm}]) \qquad (21a)$$

$$r > R \qquad H_2(r,t) = adHf_2K_o[r/\lambda_{tp}]/K_o[R/\lambda_{tp}] \qquad (21b)$$

with the following expressions for the parameters:

$$a = d_m\coth(d/\sqrt{2c'_mt})\ F_m/\sqrt{2c'_mt} \qquad (22a)$$

$$F_i = 1/(b_id_i^2/2c'_it + (d_i/\sqrt{2c'_it})\ \coth(d/\sqrt{2c'_it})) \qquad (22b)$$

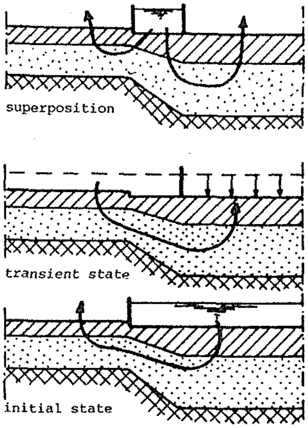
$$b_i = c'_iK_iD_i/c_iK'_id_i \qquad (22c)$$

$$\lambda_{ti} = \lambda_i\sqrt{F_i}\ ;\ \lambda_i = \sqrt{K_iD_id_i/K'_i} \qquad (22d)$$

$$f_2 = 1/(1+(KD/\lambda_t)_p(\lambda_t/KD)_m(K_1[R/\lambda_{tp}]/K_o[R/\lambda_{tp}])(I_o[R/\lambda_{tm}]/I_1[R/\lambda_{tm}]) \qquad (22e)$$

This solution presents the transient stage. A sudden drawdown has been imposed in the new polder. In reality the reclamation will not proceed so fast. The method allows to include other conditions. When the reclamation takes place over a period Δt linearly in time the solution applies with $2tdH/\Delta t(1-\exp(-\Delta t/2t))$ for dH .

The transient stage solution reveals similarity to the initial solution but the leakage is a function of time and the actual geometry. Because the processes involved are linear, the principle of superposition is valid and both stages added provides the final solution, i.e. superposition of equa-



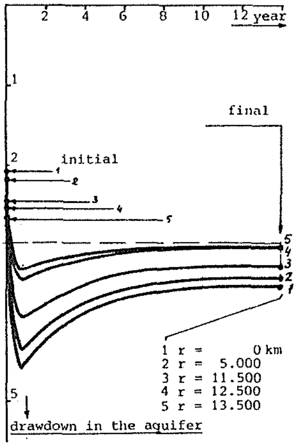


FIG. 6 Principle of superposition

FIG. 7 Some results of the model INPOLDER for the reclamation of the Markerwaard

tions (15) and (21) provides the complete description of geo-hydrological effects due to the reclamation of the polder Markerwaard in Holland. The resulting equations have been converted into a numeric computer code: INPOLDER. The program has been used to study the environmental impact due to the reclamation of the Markerwaard (Hannink, 1984).

Verification of the applied method

The applied method has been evaluated using a verification with a fully-coupled model, the SPONS code, simulating axi-symmetrical linear-elastical consolidation, which models the horizontal and vertical deformation, and porous flow. The situation concerns a homogeneous leaky aquifer in a lake. The boundary condition imposed is a sudden increase of the free watertable within a circular area: r < R, similar to the transient stage problem for Markerwaard polder. The induced piezometric head change in the aquifer is compared with the results obtained using the semi-coupled model INPOLDER.

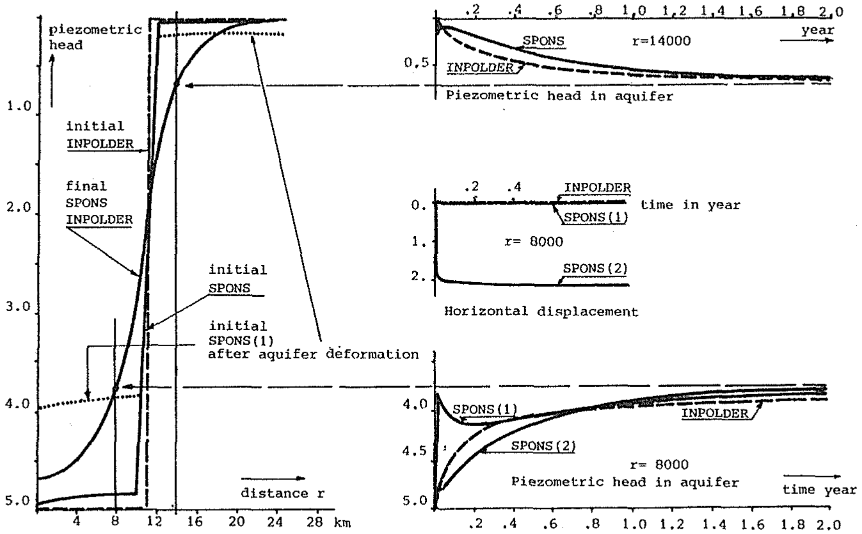


Fig. 8 Model comparison: semi-coupled INPOLDER and fully-coupled SPONS

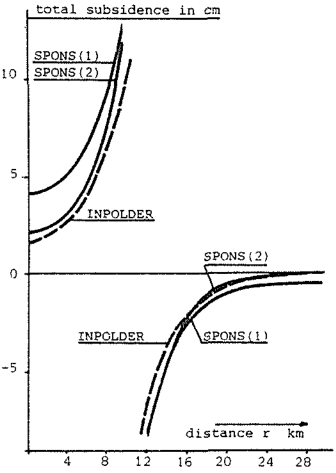


FIG. 9 Final subsidence

Some results are presented in figure 8 and 9. The initial pore-pressures are strongly affected by the adopted boundary conditions: SPONS(1) friction-free bottom, SPONS(2) fixed bottom.
This has an enormous effect on the horizontal displacements and piezometric head, particularly in the area: r < R. The results of SPONS and INPOLDER show a fair agreement. The final subsidence including compaction and consolidation, matches well for SPONS and INPOLDER.

Conclusion

To simulate regional subsidence simple models are very useful, if they include the essential phenomena: transient compaction/consolidation in a coupled way. Horizontal deformations are to be simulated with advanced models.

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FORMATION COMPACTION ASSOCIATED WITH THERMAL COOLING IN GEOTHERMAL RESERVOIRS

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Abstract

Concepts from the Theory of Interacting Continua are employed to develop constitutive relations for thermoelastic fluid-saturated porous media; these constitutive relations are shown to be equivalent to those of Biot in the isothermal limit. For uniaxial compaction, the constitutive theory is simplified to give formation compaction as a function of changes in fluid pressure and temperature. The formation compaction due to thermal cooling is equal to $3h_{\eta}$ $[(1+\nu)/3(1-\nu)]$ ΔT.

Introduction

The isothermal consolidation theory has been traditionally used in groundwater and petroleum engineering to analyze formation compaction due to fluid withdrawal. Unlike groundwater and petroleum reservoirs, however, geothermal reservoirs may undergo significant temperature changes either due to cold-fluid injection or due to production-induced flashing. In these instances, the isothermal theory is likely to be inadequate for modeling formation compaction. In this paper, we employ concepts from the theory of interacting continua (TINC) to develop constitutive relations for thermoelastic fluid-saturated earth media. TINC has previously been used by several authors to treat both infinitesimal (see e.g. Biot, 1956; Hsieh and Yew, 1973) and finite (see e.g. Morland, 1972; Garg and Nur, 1973; Garg *et al*. 1975; Carroll, 1980) deformations of rock aggregates. After outlining the basic TINC definitions, we discuss the constitutive relations for both dry (pore pressure = 0) and fluid-saturated porous media. Relationships between porosity, confining pressure, pore pressure and temperature are given next. This is followed by a discussion of the effective stress law for stress-strain response. Finally, we consider uniaxial compaction of a geologic medium due to changes in fluid pressure and temperature.

Mathematical Preliminaries

We denote by i = r and i = f the rock grain and the pore fluid, respectively. The mass of constituent i per unit volume of rock-fluid composite is called its partial density ρ^i and the total mass per unit volume of composite ρ is given by:

$$\rho = \sum_{i} \rho^{i} \qquad i = r,f \tag{1}$$

The total stress $\underset{\sim}{\sigma}$ associated with a unit area of the composite can similarly be decomposed into partial stresses $\underset{\sim}{\sigma}^i$ associated with each component i.

$$\underset{\sim}{\sigma} = \sum_i \underset{\sim}{\sigma}^i \qquad i = r,f \tag{2}$$

Partial stresses $\underset{\sim}{\sigma}^i$ and partial (bulk) densities ρ^i are related to "intrinsic stresses" $\underset{\sim}{\sigma}^{ie}$ and "intrinsic densities" ρ^{ie} through the relations:

$$\underset{\sim}{\sigma}^i = n^i \underset{\sim}{\sigma}^{ie} \tag{3a}$$

$$\rho^i = n^i \rho^{ie}, \tag{3b}$$

where

$$n^r = 1 - \phi, \tag{3c}$$

$$n^f = \phi, \tag{3d}$$

and ϕ is the porosity. In Eq. (3), it is assumed that area and volume fractions are the same for the i^{th} constituent.

The stresses $\underset{\sim}{\sigma}^i$ and $\underset{\sim}{\sigma}^{ie}$ may be decomposed into hydrostatic (p^i and p^{ie}) and deviatoric ($\underset{\sim}{S}^i$ and $\underset{\sim}{S}^{ie}$) parts as follows:

$$\underset{\sim}{\sigma}^i = - p^i \underset{\sim}{I} + \underset{\sim}{S}^i \tag{4a}$$

$$\underset{\sim}{\sigma}^{ie} = - p^{ie} \underset{\sim}{I} + \underset{\sim}{S}^{ie} \tag{4b}$$

$$p^i = n^i p^{ie}, \; \underset{\sim}{S}^i = n^i \underset{\sim}{S}^{ie} \tag{4c}$$

Since the fluid cannot sustain any shear stresses, it follows that $\underset{\sim}{S}^f = \phi \underset{\sim}{S}^{fe} = 0$.

Partial volumetric strains (positive in tension) are defined as

$$\varepsilon^i = \left(\rho_0^i \middle/ \rho^i\right) - 1 \qquad i = r,f \tag{5a}$$

where the subscript 0 denotes the initial value of the subscripted variable. Partial volumetric strain for the rock ε^r is often denoted in the literature as the bulk volumetric strain. Partial deviatoric strain $\underset{\sim}{e}^i$ is given by:

$$\underset{\sim}{e}^i = \underset{\sim}{\varepsilon}^i - (\varepsilon^i/3) \underset{\sim}{I}, \tag{5b}$$

where

$$\underset{\sim}{\varepsilon}^i = \frac{1}{2} [\nabla \underline{u}^i + (\nabla \underline{u}^i)^t] \tag{5c}$$

and $\underline{u}^i$ denotes the displacement field for the i^{th} component.

Constitutive Relations

In the rock-fluid mixture, only the rock matrix can sustain shear stresses, and the rock partial shear stresses and strains ($\underset{\sim}{S}^r$ and $\underset{\sim}{e}^r$) equal the composite shear stresses ($\underset{\sim}{S}$) and strains ($\underset{\sim}{e}$). In this case, one can directly postulate a shear law for the entire mixture. We shall assume that the shear response is governed by Hooke's law:

$$\underset{\sim}{S} = 2\, \mu_p \underset{\sim}{e} \tag{6}$$

where μ_p denotes the shear modulus of the porous rock. To relate partial hydrostatic stresses p^i to partial volumetric strain ε^i, it is necessary to introduce intrinsic volumetric strains ε^{ie}.

$$\varepsilon^{ie} = \rho_o^{ie}/\rho^{ie} - 1 = \left(n^i/n_o^i\right)(1+\varepsilon^i) - 1 \qquad i = r,f. \tag{7}$$

A mixture model for hydrostatic stress is formulated by assuming that the pressure law (p versus ε and T relationship) for each isolated component (rock and fluid) may be used to relate p^{ie}, ε^{ie} and T. Thus, if the pressure law for the ith component as a single continuum is given by

$$p = f\ (\varepsilon,T) \tag{8a}$$

then the pressure law for the ith component within the mixture is expressed by

$$p^i = n^i\ f\ (\varepsilon^{ie}, T^i) \tag{8b}$$

Constitutive relations can now be written for dry (pore pressure = 0) and fluid-saturated porous rocks. We assume that the rock grain and the fluid behave in a linear thermoelastic manner.

Dry Porous Rock: The relations for dry porous rock are as follows:

$$\underset{\sim}{\sigma} = - P_c \underset{\sim}{I} + \underset{\sim}{S} \tag{9a}$$

$$P_c = p^r \tag{9b}$$

$$p^r = n^r p^{re} = - (1-\phi)\ K_s\ (\varepsilon^{re} -3\, \eta_s\ T) \tag{9c}$$

$$\varepsilon^{re} = \frac{1-\phi}{1-\phi_o}\ (1 + \varepsilon^r)\ -1 \tag{9d}$$

Here K_s (η_s) denotes the bulk modulus (coefficient of linear expansion) for the rock grain.

Saturated Wet Rock: The relations for saturated wet rock are as follows:

$$\underset{\sim}{\sigma} = - P_c \underset{\sim}{I} + \underset{\sim}{S} \quad (10a)$$

$$P_c = p^r + p^f \quad (10b)$$

where p^r is given by (9c) - (9d) and p^f is evaluated from:

$$p^f = n^f p^{fe} = -\phi K_f (\varepsilon^{fe} - 3 \eta_f T) \quad (10c)$$

$$\varepsilon^{fe} = \frac{\phi}{\phi_0} (1 + \varepsilon^f) - 1 \quad (10d)$$

Here K_f (η_f) denotes the bulk modulus (coefficient of linear expansion) for the fluid, and we have assumed that the rock grain and the fluid are in local thermal equilibrium. Also note that p^f is related to the actual fluid pressure P_f through the relation:

$$p^f = \phi p^{fe} = \phi P_f \quad (10e)$$

The constitutive relations (7), (9) and (10) are complete only when porosity ϕ is prescribed. For a dry porous rock, porosity ϕ may be expressed as a function of the confining pressure P_c (or equivalently ε^r) and temperature T. It is assumed here that the porosity ϕ is independent of shear stresses. Experimental data at sufficiently high shear stress levels indicate that both hydrostatic and shear stresses depend on both volumetric and shear strains. The present model may be extended to such cases by including the dependence of ϕ on shear stresses (or strains). For the present purposes it is not necessary to entertain this more general case. For the saturated rock, porosity ϕ is postulated to depend on both P_c and P_f (or equivalently on ε^r and ε^f) in addition to T.

Porosity - Pressure/Temperature Relationship for Dry Rock

We postulate that porosity ϕ is a linear function of confining pressure P_c and temperature T.

$$\phi = \phi_0 [1 + \alpha P_c + \beta T] \quad (11)$$

where α and β are as yet unspecified constants. We shall next express α and β in terms of readily measurable rock properties. We assume that the bulk rock (i. e., porous rock) behaves in a linear thermoelastic manner

$$P_c = - K (\varepsilon^r - 3 \eta T) \quad (12a)$$

where K (η) denotes the bulk modulus (coefficient of linear thermal expansion) of the bulk rock.

$$-K = \left(\frac{\partial P_c}{\partial \varepsilon^r} \right)_T , \quad 3 K\eta = \left(\frac{\partial P_c}{\partial T} \right)_{\varepsilon^r} . \quad (12b)$$

Differentiating Eq. (11) with respect to ε^r and T and using Eq. (12), we have:

$$\left(\frac{\partial\phi}{\partial\varepsilon^r}\right)_T = - \phi_o \ \alpha K \tag{13a}$$

$$\left(\frac{\partial\phi}{\partial T}\right)_{\varepsilon^r} = \phi_o \ [3\alpha \ K\eta \ + \ \beta] \tag{13b}$$

In subsequent analysis, we shall require $(\partial\varepsilon^{re}/\partial\varepsilon^r)_T$ and $(\partial\varepsilon^{re}/\partial T)_{\varepsilon^r}$. Differentiating Eq. (9d) with respect to ε^r and T, ignoring terms of O (ε), and using Eq. (13), we obtain:

$$\left(\frac{\partial\varepsilon^{re}}{\partial\varepsilon^r}\right)_T = 1 + \frac{1}{1-\phi_o} \ (\phi_o \ \alpha K) \tag{14a}$$

$$\left(\frac{\partial\varepsilon^{re}}{\partial T}\right)_{\varepsilon^r} = - \frac{1}{1-\phi_o} \ \phi_o \ (3\alpha K\eta + \beta) \tag{14b}$$

To determine α, we differentiate Eq. (9c) with respect to ε^r, ignore terms of O(ε), and use Eq. (14a).

$$\left(\frac{\partial P_c}{\partial\varepsilon^r}\right)_T = - K_s \ (1-\phi_o) \left[1 + \frac{\phi_o\alpha K}{1-\phi_o}\right]$$

Substituting for $(\partial P_C/\partial\varepsilon^r)_T$ from Eq. (12b) into the above expression, it follows that

$$\alpha = \frac{1}{\phi_o} \left[\frac{1}{K_s} - \frac{1-\phi_o}{K}\right] \tag{15}$$

To obtain β, we differentiate Eq. (9c) with respect to T, ignore terms of O(ε) and use Eq. (14b).

$$\left(\frac{\partial P_c}{\partial T}\right)_{\varepsilon^r} = (1-\phi_o) \ K_s\left[\frac{\phi_o}{1-\phi_o} \ (3\alpha K\eta + \beta) + 3\eta_s\right]$$

Combining the above expression with Eqs. (12c) and (15), there follows

$$\beta = 3\left(\frac{1 - \phi_o}{\phi_o}\right) (\eta - \eta_s) \tag{16}$$

Substituting from Eqs. (15) and (16) into Eq. (11), we have for the dry porous rock:

$$\phi = \phi_o + [1/K_s - (1/\phi_o)/K]\ P_c + 3(1-\phi_o)\ (\eta-\eta_s)T \tag{17}$$

Porosity - Pressure/Temperature Relationship for Saturated Rock

Porosity ϕ for the fluid-saturated rock can be regarded as a linear function of P_c, P_f and T (or alternatively of ε^r, ε^f and T), i.e.

$$\phi = \phi_o\ [1 + A\ P_c + B\ P_f + CT] \tag{18}$$

where A, B and C are as yet unspecified constants. We shall presently prove that

$$A = - B \tag{19}$$

Differentiating Eqs. (9d) and (10d) with respect to ε^r and ε^f, and ignoring terms of $O(\varepsilon)$, gives:

$$\left(\frac{\partial \varepsilon^{re}}{\partial \varepsilon^{r}}\right)_{\varepsilon^f,T} = 1 - \frac{1}{1-\phi_o} \left(\frac{\partial \phi}{\partial \varepsilon^{r}}\right)_{\varepsilon^f,T} \tag{20a}$$

$$\left(\frac{\partial \varepsilon^{re}}{\partial \varepsilon^{f}}\right)_{\varepsilon^r,T} = - \frac{1}{1-\phi_o} \left(\frac{\partial \phi}{\partial \varepsilon^{f}}\right)_{\varepsilon^r,T} \tag{20b}$$

$$\left(\frac{\partial \varepsilon^{fe}}{\partial \varepsilon^{r}}\right)_{\varepsilon^f,T} = \frac{1}{\phi_o} \left(\frac{\partial \phi}{\partial \varepsilon^{r}}\right)_{\varepsilon^f,T} \tag{20c}$$

$$\left(\frac{\partial \varepsilon^{fe}}{\partial \varepsilon^{f}}\right)_{\varepsilon^r,T} = 1 + \frac{1}{\phi_o} \left(\frac{\partial \phi}{\partial \varepsilon^{f}}\right)_{\varepsilon^r,T} \tag{20d}$$

We next differentiate Eqs. (9c) and(10c) with respect to ε^r and ε^f, ignore terms of $O(\varepsilon)$, and utilize Eqs. (20) to obtain:

$$\left(\frac{\partial p^{r}}{\partial \varepsilon^{r}}\right)_{\varepsilon^f,T} = - (1-\phi_o)\ K_s \left[1 - \frac{1}{1-\phi_o} \left(\frac{\partial \phi}{\partial \varepsilon^{r}}\right)_{\varepsilon^f,T}\right] \tag{21a}$$

$$\left(\frac{\partial p^{r}}{\partial \varepsilon^{f}}\right)_{\varepsilon^r,T} = K_s \left(\frac{\partial \phi}{\partial \varepsilon^{f}}\right)_{\varepsilon^r,T} \tag{21b}$$

$$\left(\frac{\partial p^{f}}{\partial \varepsilon^{r}}\right)_{\varepsilon^f,T} = - K_f \left(\frac{\partial \phi}{\partial \varepsilon^{r}}\right)_{\varepsilon^f,T} \tag{21c}$$

$$\left(\frac{\partial p^{f}}{\partial \varepsilon^{f}}\right)_{\varepsilon^r,T} = - \phi_o\ K_f \left[1 + \frac{1}{\phi_o} \left(\frac{\partial \phi}{\partial \varepsilon^{f}}\right)_{\varepsilon^r,T}\right] \tag{21d}$$

We also have from Eqs. (10b), (10c) and (21):

$$\left(\frac{\partial P_c}{\partial \varepsilon^r}\right)_{\varepsilon^f,T} = \left(\frac{\partial [p^r + p^f]}{\partial \varepsilon^r}\right)_{\varepsilon^f,T} = -(1-\phi_0)\, K_s + (K_s - K_f)\left(\frac{\partial \phi}{\partial \varepsilon^r}\right)_{\varepsilon^f,T} \tag{22a}$$

$$\left(\frac{\partial P_c}{\partial \varepsilon^f}\right)_{\varepsilon^r,T} = \left(\frac{\partial [p^r + p^f]}{\partial \varepsilon^f}\right)_{\varepsilon^r,T} = -\phi_0\ K_f + (K_s - K_f)\left(\frac{\partial \phi}{\partial \varepsilon^f}\right)_{\varepsilon^r,T} \tag{22b}$$

$$\left(\frac{\partial P_f}{\partial \varepsilon^r}\right)_{\varepsilon^f,T} = \frac{1}{\phi_0}\left(\frac{\partial p^f}{\partial \varepsilon^r}\right)_{\varepsilon^f,T} = -\frac{K_f}{\phi_0}\left(\frac{\partial \phi}{\partial \varepsilon^r}\right)_{\varepsilon^f,T} \tag{22c}$$

$$\left(\frac{\partial P_f}{\partial \varepsilon^f}\right)_{\varepsilon^r,T} = \frac{1}{\phi_0}\left(\frac{\partial p^f}{\partial \varepsilon^f}\right)_{\varepsilon^r,T} = -K_f\left[1 + \frac{1}{\phi_0}\left(\frac{\partial \phi}{\partial \varepsilon^f}\right)_{\varepsilon^r,T}\right] \tag{22d}$$

Next, we shall obtain expressions for

$$\left(\frac{\partial \phi}{\partial \varepsilon^r}\right)_{\varepsilon^f,T} \quad \text{and} \quad \left(\frac{\partial \phi}{\partial \varepsilon^f}\right)_{\varepsilon^r,T}.$$

Differentiating Eq. (18) with respect to ε^r and ε^f and combining with Eq. (22), we have

$$\left(\frac{\partial \phi}{\partial \varepsilon^r}\right)_{\varepsilon^f,T} = -\phi_0\ (1-\phi_0)\ A\ K_s/X \tag{23a}$$

$$\left(\frac{\partial \phi}{\partial \varepsilon^f}\right)_{\varepsilon^r,T} = -\phi_0\ K_f\ [A\ \phi_0 + B]/X, \tag{23b}$$

where

$$X = 1 - \phi_0\ A\ (K_s - K_f) + B\ K_f.$$

Garg and Nur [1973] showed that:

$$\left(\frac{\partial p^r}{\partial \varepsilon^f}\right)_{\varepsilon^r,T} = \left(\frac{\partial p^f}{\partial \varepsilon^r}\right)_{\varepsilon^f,T} \tag{24}$$

Eq. (19) follows upon substituting from Eqs. (21b), (21c) and (23) into Eqs. (24), Therefore,

$$\phi = \phi_o \, [1 + A \, (P_c - P_f) + CT] \tag{18'}$$

To determine A and C, we note that when $P_f = 0$, the porosity ϕ given by Eq. (18') should be the same as it is when the rock is dry [Eq. (17)]. This yields

$$A = \frac{1}{\phi_o} \left[\frac{1}{K_s} - \frac{1-\phi_o}{K} \right] \tag{25a}$$

and

$$C = \frac{3(1-\phi_o)}{\phi_o} \quad (\eta - \eta_s) \quad . \tag{25b}$$

Thus for the saturated rock case, we have:

$$\phi = \phi_o + [1/K_s - (1-\phi_o)/K] \, (P_c - P_f) + 3(1-\phi_o) \, (\eta - \eta_s)T \tag{26}$$

Comparison of Eqs. (17) and (26) shows that the effective pressure for porosity $<P_{eff}>_\phi$ is given by:

$$<P_{eff}>_\phi = P_c - P_f. \tag{27}$$

Effective Stress Law for Stress-Strain Response

Substituting for ϕ from Eq. (26) into Eq. (9d) and ignoring terms of $O(\varepsilon^2)$, we have:

$$\varepsilon^{re} = \varepsilon^r - \frac{1}{1-\phi_o} \Big\{ [1/K_s - (1-\phi_o)/K] \quad (P_c - P_f)$$

$$+ \, 3 \, (1 - \phi_o) \, (\eta - \eta_s) \; T \Big\} \tag{28}$$

Combining Eqs. (10b), (10c), (9c) and (28), we obtain

$$P_c - \left(1 - \frac{K}{K_s} \right) P_f = -K \left[\varepsilon^r - 3\eta T \right] \tag{29}$$

Comparing Eqs. (12a) and (29) and utilizing Eq. (10a), we see that the stress-strain response of the fluid-saturated rock is governed by the effective stress $<\sigma>_\varepsilon$:

$$\begin{aligned} <\underset{\sim}{\sigma}>_\varepsilon &= \underset{\sim}{\sigma} + (1-K/K_s) \, P_f \\ &= - \, [P_c - (1-K/K_s) \, P_f] \, \underset{\sim}{I} + \underset{\sim}{S} \\ &= K \, [\varepsilon^r - 3\eta T] \, \underset{\sim}{I} + \underset{\sim}{S} \end{aligned} \tag{30}$$

The last result is identical with that previously derived by Biot and Willis (1957) and by Garg and Nur (1973) for the isothermal case.

In the isothermal limit, the present constitutive theory involves two rock bulk moduli (drained bulk modulus K, intrinsic or unjacketed bulk modulus, K_s), porous rock shear modulus μ_p, and fluid bulk modulus K_f. These constitutive parameters are exactly the same as those used by Biot and Willis (1957) to characterize the rock-fluid composite.

Uniaxial Compaction of a Geologic Medium Due to Fluid Withdrawal

Fluid withdrawal from geothermal reservoirs may be accompanied by drops in both the fluid pressure and the fluid/rock temperature. It is therefore necessary to account for both pressure and temperature changes in subsidence/compaction calculations. To illustrate the last remark, let us consider the case of uniaxial compaction. For uniaxial strain case, we have:

$$\varepsilon^r = \varepsilon_x$$

$$e^r = 2/3\ \varepsilon_x \quad . \qquad (31)$$

Substituting from Eqs. (31) into Eqs. (6) and (30), we obtain:

$$\begin{aligned} \langle\sigma_x\rangle_\varepsilon &= \sigma_x + (1 - K/K_s)\ P_f \\ &= (K + 4\mu_p/3)\ \varepsilon_x - 3K\eta T \end{aligned} \qquad (32)$$

A common assumption in subsidence analysis is

$$\dot{\sigma}_x = 0 \qquad (33)$$

Equation (33) implies that the mass of fluid withdrawn is small so that the overburden remains essentially constant. Substituting from Eq. (33) into Eq. (32), we get the following expression for uniaxial strain ε_x:

$$\dot{\varepsilon}_x = \Delta h/h = C_m\ \dot{P}_f + C_T\ \dot{T} \qquad (34)$$

where

$$C_m = (1\text{-}K/K_s)/(K\text{+}4\mu_p/3),\ C_T = 3\eta K/(K\text{+}4\mu_p/3)$$

and h is the formation thickness.

The first term in Eq. (34) is identical with that appearing in the usual expressions employed to evaluate the compaction of oil/groundwater reservoirs. Groundwater/oil systems are usually treated as isothermal systems, and therefore thermal effects (e.g. second term in Eq. (34)) have not been considered in previous analyses. It is, however, apparent from Eq. (34) that for geothermal systems (where temperature drop may be significant), thermally induced compaction may be quite important. From Eq. (34), the formation compaction due to thermal cooling is given by

$$\Delta h_{thermal} = 3h\eta[K/(K+4\mu_p/3)]\ \Delta T$$

$$= 3h\eta[(1+\nu)/3(1-\nu)]\ \Delta T$$

where ν is the drained Poisson's ratio and ΔT is the temperature drop. With $\nu = 0.2$, $3\eta = 5 \times 10^{-5}/°C$, $h = 1000m$ and $\Delta T = 40°C$, we have $\Delta h_{thermal} \sim 1$ m. A compaction of the order of one meter can have serious consequences for the integrity of reinjection wells.

Acknowledgment: This work was performed for the University of Texas at Austin under U. S. Department of Energy contract No. E(40-1) - 5040.

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A PREDICTION OF SURFACE SUBSIDENCE CAUSED BY LOWERING THE WATER TABLE IN DOLOMITE

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Abstract

The proposed dewatering of a gold mine will result in the lowering of the water table over a large area underlain by dolomite. Statistical techniques have been used to predict the probability of subsidence under certain surface structures, and the magnitude of subsidence has been related to categories of damage for the different buildings.

Introduction

The gold mines of the Far West Rand, South Africa, extract ore from quartzites at depths up to 3 000 metres below the ground surface. The gold-bearing strata are overlain by chert and dolomite. Water from cavities and fissures in the dolomite passes through tension faults into the mine workings.

At a mine with which this paper is concerned (hereafter referred to as Mine A), water pumped from the workings is returned to the groundwater, to prevent a lowering of the water-table. Previous dewatering at other mines has resulted in surface sinkholes and subsidence associated with a lowered water-table. Unfortunately, the monthly volume of pumped water has increased steadily from 2 million kl in 1973 to 3,5 million kl in 1983, and appears to be growing by 25 000 kl per month. The increase is caused by the gradual clearing of channels by the inflow.

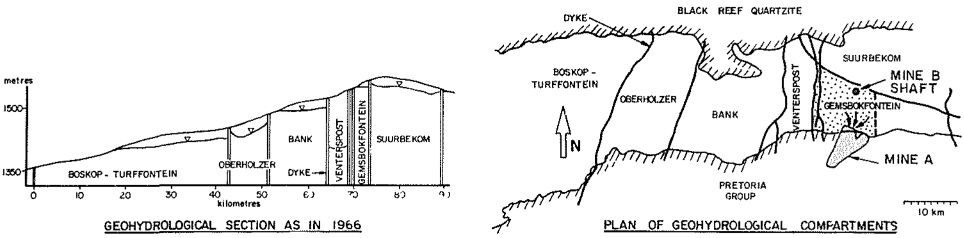
Two methods of avoiding the increasing cost of circulating the water are being considered. The first is the creation of a grout barrier. The second is the disposal of the water outside the groundwater compartment, resulting in a lowering of the water-table and a consequent reduction of pumped volumes.

This paper describes the investigation of potential damage to surface structures at a mine (hereafter referred to as Mine B) in the vicinity of Mine A where dewatering will take place. The Mine B shaft is 3 km from Mine A, and the water table in the shaft area would be affected by the proposed dewatering.

Geology

The Chuniespoort Group dolomite and chert of the Wonderfontein Valley in the Far West Rand overlie the gold-bearing strata of the Witwatersrand Supergroup. Outliers of Karoo Sediments occupy depressions in the dolomite. The dolomite formation is divided into groundwater compartments by intrusive dykes, and water spills from one compartment into the next through 'eyes'. The mine to be dewatered (Mine A) and the mine surface structures with which this paper is concerned (Mine B) are within the Gemsbokfontein Compartment (Fig 1).

The dolomites are noted for their irregular weathering, and for the surface subsidence and sinkholes precipitated by water movements.



GEOHYDROLOGICAL SECTION AS IN 1966

PLAN OF GEOHYDROLOGICAL COMPARTMENTS

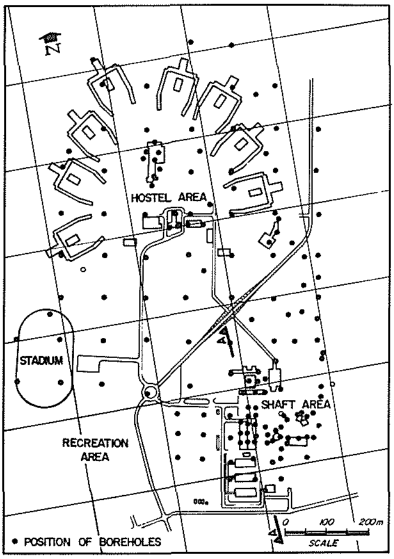
FIG. 1 Section and plan of groundwater compartments of Far West Rand
(Adapted from Enslin & Kriel, 1968)

At the Mine B shaft area, approximately 130 percussion boreholes were drilled into bedrock prior to the sinking of the shaft and the construction of surface structures. The surface structures and borehole positions are shown in Fig 2. The structures were positioned after a gravity survey was carried out. The survey indicated a zone of deep weathering in the south-west of the site. The geological sequence is typically as follows;

0 - 8 m	Transported red sand and clay
8 m - 40 m	Brown clay, residual from dolomite
40 m - 50 m	Wad or wad with clay
50 m - 53 m	Slightly weathered dolomite
From 53 m	Unweathered dolomite

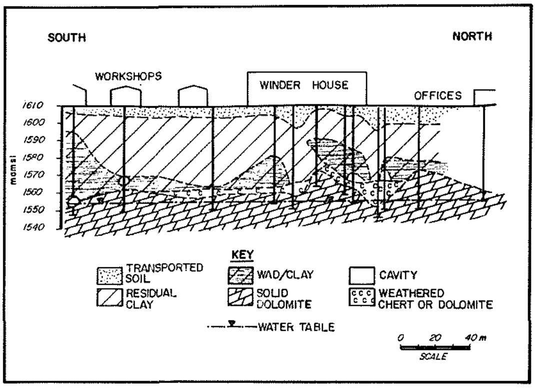
The water table depth is 50 m below ground surface. Wad is a very compressible product of the weathering and leaching of dolomite.

A typical geological section is shown in Fig 3. Cavities were noted in many exploratory boreholes, but borehole cameras have indicated that, generally, these apparent cavities contain interbedded layers of wad and thin chert bands.



◁FIG. 2 Plan of structures and borehole positions

FIG. 3 Typical geological section ▽

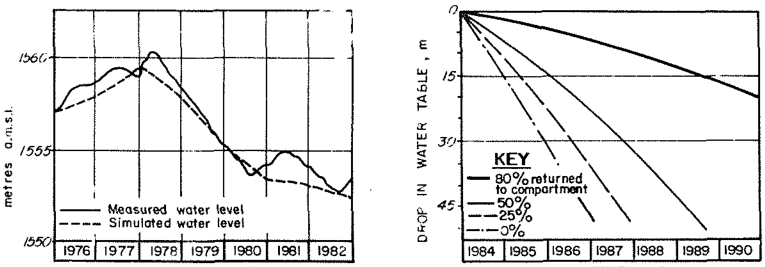


Prediction of water table response to dewatering

The magnitude of potential subsidence will depend upon the extent of the drop in the water-table, and the speed with which it occurs. However, when the water-table had dropped below the zones of weathering and dolomitic cavities, further water-table lowering will not cause subsidence.

To predict the water-table response at Mine B to dewatering at Mine A, it was assumed that the western section of the Gemsbokfontein Compartment is surrounded by impermeable barriers, with inflow from rainfall and artificial recharge, and outflow from pumping at Mine B. The geohydrological compartment is shown shaded in Fig 1. The dashed line represents a postulated extension of a known dyke. This extension has been suggested by evidence from remote sensing and groundwater levels (Fleisher, 1981).

Dewatering (ie, water discharged outside the compartment) was carried out between 1977 and 1982. Therefore, using the known water volumes, it was possible to compare a predicted draw-down to that actually measured at an observation well 1,75 km from the Mine B shaft. A recharge value of 13% of annual rainfall was used (Fleisher, 1981), and an average transmissivity of 2 000 kl/day/m was computed from results of pump tests in an adjacent compartment.



a) SIMULATED WATER LEVEL 1976-82

b) PREDICTED EFFECT OF FUTURE DEWATERING

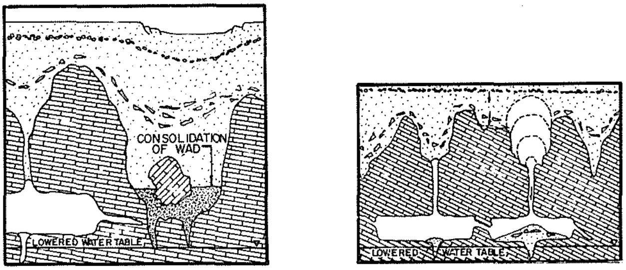
FIG. 4 Water table movement

The good agreement (Fig 4a) between predicted and actual values engenders confidence in the use of the model to predict water table levels at Cooke 3 during the proposed dewatering. Water table movements were predicted for conditions where various proportions of the pumped water are returned to the groundwater compartment (Fig 4b). The Theis non-equilibrium solution was used, and the effect of impermeable boundaries was dealt with by using the method of images (Ferris et al, 1962).

The subsidence hazard

The mechanisms of doline and sinkhole formation as perceived by Jennings (1966) & Brink (1979) are described pictorially in Fig 5. The effect of a drop in the watertable is to accelerate sinkhole formation, when the water-table is in a critical initial position relative to the potential sinkhole. In the case of doline formation, settlement is caused by an increase in the effective stress when the water-table drops through a compressible layer of wad.

Clearly, the extent to which a change in water-table level causes subsidence depends upon the position of the water-table relative to problem zones.



DOLINE SINKHOLE

FIG. 5 Mechanisms of doline and sinkhole formation (Brink, 1979)

Considerable experience has been accumulated of the subsidence caused by dewatering in the Far West Rand. Dewatering of the West Driefontein Mine in the Oberholzer Compartment, the Venterspost Mine in the Venterspost Compartment, and the West Driefontein Mine in the Bank Compartment (the West Driefontein Mine had penetrated the Bank Compartment through the Bank Dyke) have resulted in extensive damage caused by sinkholes and dolines. The following empirical guidelines have been developed;

- Sinkholes occur most frequently where the depth of the original water table is less than 30 m. In areas where the depth of the original water-table is between 30 and 60 m, relatively few sinkholes have occurred (Kleywegt, 1980).
- Roux (1981) suggests that the ratio depth to wad/thickness of wad is an indication of potential subsidence as follows;

Ratio	10	no subsidence
	4 to 10	safe if surface water controlled
	1 to 4	high risk
	1	very high risk

- In the Far West Rand, about 10 to 15% of the area affected by the lowering of the water-table has been affected by slow ground settlement (Kleywegt, 1980)
- It has been found that an overburden cover of 15 m will usually be sufficient to prevent the formation of sinkholes (Kleywegt, 1980).

From a knowledge of the geological and water conditions at Cooke 3, it was concluded that there is a low probability of sinkhole formation resulting from dewatering. A simplified model of subsidence resulting from the compression of soft zones of wad was adopted for the prediction of settlements at the ground surface.

The model of subsidence

It was considered that settlement will only occur if the effective vertical stress in a layer of soft material increases because the water-table drops through it. Further, it was conservatively assumed that the material above the unweathered dolomite is compressible wad of a sufficient lateral extent that the magnitude of surface subsidence will be equal to the reduction in thickness of the wad layer.

If the water-table movement is slow, complete drainage will occur, but if it is fast, 'hanging' water will increase the effective stress above its final equilibrium value. The curves of Fig 6 were obtained using a compression index of 1 for wad, derived from results reported by Brink (1979). The upper curve, representing the case for a rapid lowering of the water table,

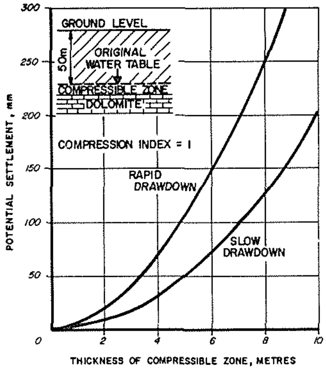


FIG. 6 Potential settlement of wad

was used in subsequent analyses. It is based on an assumption that the increase in effective stress will be doubled by rapid drawdown.

Calculation of potential subsidence

Fig 7 shows a histogram of the depth below surface of unweathered dolomite taken from borehole data. The data was analysed using the geostatistical method of Universal Kriging (Matheron, 1971) in the computer program GEOPAK 2 (Grady, 1983). Kriging allows one to take account of the fact that a measured depth at one point must be related to that of another nearby point, the dependence diminishing for points further apart. A semi-variogram is used to define the relationship. One half of the variance of the difference in the depth of unweathered dolomite for holes a certain distance apart was plotted against that distance (Fig 8). The straight line drawn through the points for boreholes up to 500 metres apart is a confirmation of interdependence.

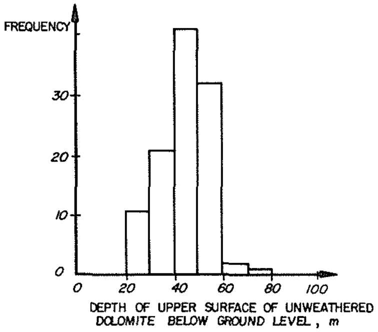


FIG. 7 Histogram of depth to unweathered bedrock

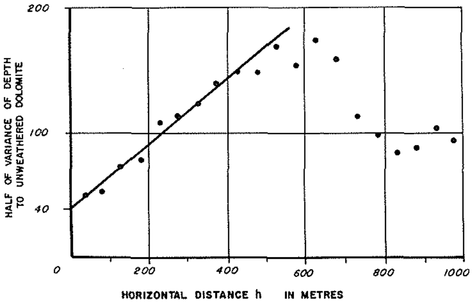
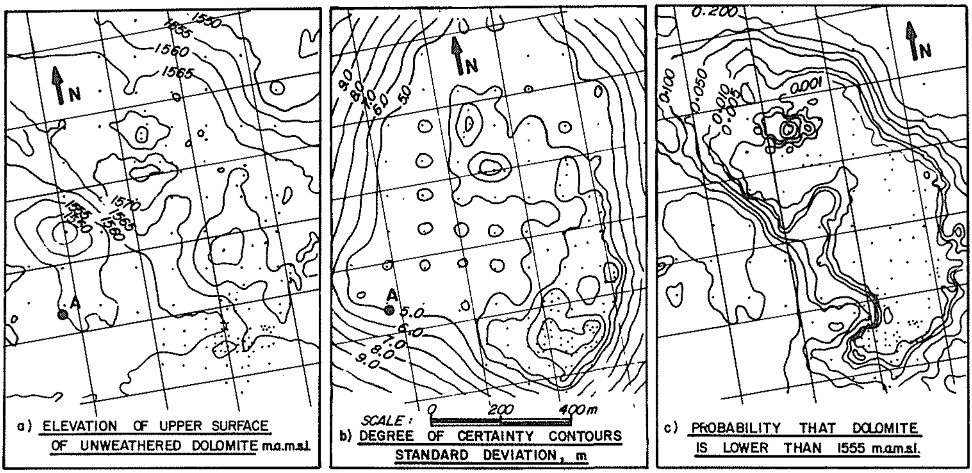


FIG. 8 Semi-variogram for depth of unweathered bedrock



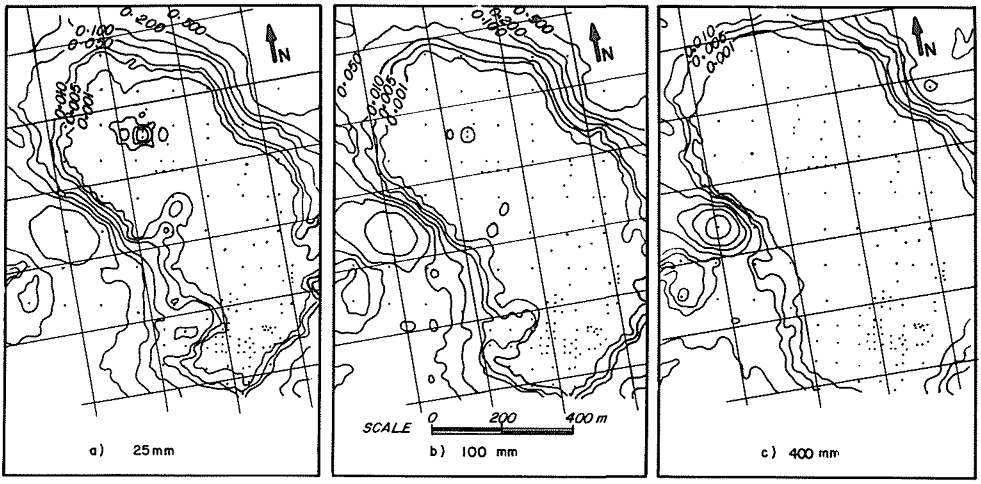
a) ELEVATION OF UPPER SURFACE OF UNWEATHERED DOLOMITE m.a.m.s.l.

b) DEGREE OF CERTAINTY CONTOURS STANDARD DEVIATION, m

c) PROBABILITY THAT DOLOMITE IS LOWER THAN 1555 m.a.m.s.l.

FIG. 9 Contours of elevation, degree of certainty, and probability obtained from geostatistical analysis

A contour plot of the depth of unweathered dolomite was produced (Fig 9) together with an associated degree of certainty plot, which gave contours of standard deviation. Block Kriging was used and the contours represent the average elevation of blocks 25 m x 25 m. This block size was conservatively chosen as the smallest area which would cause a surface settlement equal to the change in thickness of a compressible layer at depth. For point A in Fig 9, the estimated elevation of 1 555 mamsl has a standard deviation of 5 m. Two standard deviations imply a degree of certainty of 97%. Therefore there is a 97% certainty that the upper surface of the unweathered dolomite at point A will be within the range (1 555 - 10) to (1 555 + 10) mamsl.



a) 25mm

b) 100 mm

c) 400 mm

FIG. 10 Contours of probability for different magnitudes of settlement

From the degree of certainty and the estimated elevations, the computer is able to produce contour plots of the probability that the unweathered dolomite is below a chosen elevation. Thus, for a water table at 1 556 mamsl a probability plot for a dolomite elevation of 1 555 mamsl gives the probability that lowering the water table will compress one metre of supposed wad, causing 4 mm of surface settlement (Fig 9). A series of such plots for different dolomite elevations (Figs 10(a), (b) and (c)) show contours of the probability that the upper surface of unweathered rock is 2½, 5 and 10 m below the water table, equivalent to settlements of 25, 100 and 400 mm, respectively, allows the derivation of the relationship between the probability of occurrence and predicted settlement for a chosen point or structure (Fig 11).

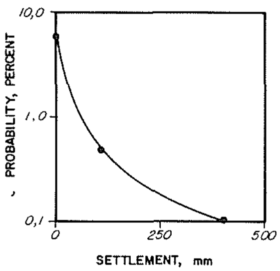


FIG. 11 Typical settlement/probability curve for individual building

Damage to structures

Table 1 was prepared for use as a measure of the effects of the predicted subsidence at Mine B. It is a simple division, but it was considered that, in this investigation, a more sophisticated set of damage criteria was unjustified. It was devised after considering the recommendations of various authors (eg Voight & Pariseau, 1970, Burland and Wroth, 1975).

TYPE OF STRUCTURE	RANGE OF TOTAL SETTLEMENT, mm		
	INSIGNIFICANT DAMAGE	DAMAGE REQUIRING REPAIRS	STRUCTURAL DAMAGE
LOAD - BEARING BRICK	0 - 5	5 - 25	> 25
CONCRETE FRAME WITH BRICK INFILL	0 - 10	10 - 50	> 50
STEEL FRAME WITH BRICK INFILL	0 - 10	10 - 75	>75
STEEL FRAME WITH STEEL CLADDING	0 - 20	20 - 75	>75

TABLE 1 Categories of building damage

The settlement required for each of the categories of damage was obtained for each surface structure, and their probability of occurence was obtained from the statistical data (eg Fig 11). The final step was to define a significant probability. In other words, at what calculated probability does the risk of a particular category of damage become significant for a particular structure.

The following significant probabilities were chosen, based upon a consideration of the method of analysis and the importance of the structures:

STRUCTURES	SIGNIFICANT PROBABILITY
Shaft structure, Winder houses, fan and duct	1,0%
Other structures	10,0%

The selection was naturally influenced by experience and by a comparison of the type of qualitative hazard plan that would have been produced manually. However, as the analytical techniques are improved by calibrating them against actual recorded movements, it should become feasible to select significant probabilities from a more direct consideration of the economic and social consequences of subsidence.

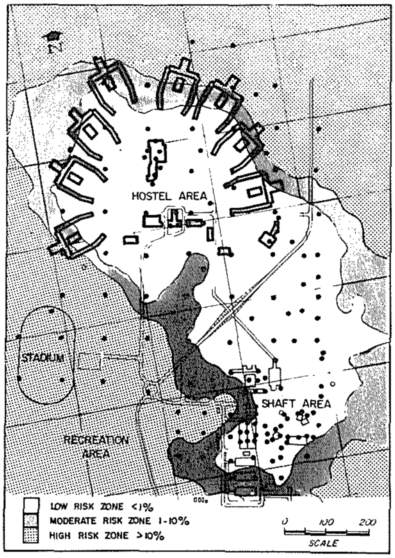


FIG. 12 Hazard plan based on Fig. 9(c)

The zoning of Fig 12 is based upon the 10% criterion for structures other than the shaft structures. It is intended as a visual indication only, but it is of interest to examine briefly how it differs from a conventional hazard plan. The conventional plan is derived from the available data, whereas the probabilistic analysis also takes account of a lack of data. In an area with sparse data, the degree of certainty is low, and the statistical probability of subsidence is therefore increased (eg compare the three plots in Fig 9).

The conclusions about potential subsidence were as follows:

- There is a low risk of sinkhole formation caused by a lowering of the water-table, but a significant risk of surface subsidence
- The risk of damage to essential shaft structures is low
- Several hostel buildings have a high risk of damage

- There is a low risk of damage to other individual structures but a high risk of damage to a small proportion of the group of structures
- Subsidence is likely in the recreation area to the south-west
- Surface subsidence may be reduced by ensuring that the water-table is lowered slowly over the first four years of dewatering
- Surface and sub-surface movement should be monitored using surface pegs and telescopic benchmarks.
- Additional exploratory holes should be drilled in critical areas where information is sparse

Discussion

The following procedure for the prediction of subsidence caused by a lowering water-table in dolomite has been described:-

(a) Derivation of simple subsidence model.
(b) Statistical prediction of subsidence and probability of occurence.
(c) Definition of critical settlement for different categories of damage and different types of structure.
(d) Definition of significant or critical probability of subsidence.
(e) Prediction of probability of different categories of damage for individual structures

A logical last step might have been the calculation of the costs of maintenance and repair, to be used when comparing the two alternative solutions of a water barrier or dewatering being considered by the Mine B.

The method is a novel approach to the prediction of potential subsidence on dolomites. The analyses involved simplifying assumptions, but these were conservative and based upon past experience. It has commonly been considered that dolomitic subsidence is so arbitrary that a prediction more sophisticated than a qualitative hazard plan is unjustified. However, the probabilistic method described here provides a numerical prediction which is not in conflict with a qualitative assessment and is a more convenient planning tool.

The depth of the water-table of 50 m, implying a low probability of sinkhole formation, allowed the adoption of a simple settlement model, but if required, more complex models, to cope with the mechanism of sinkhole formation, could be incorporated.

Conclusions

The statistical and probabilistic approach to the prediction of dolomitic subsidence provides a numerical solution which is more amenable to an objective assessment than is a qualitative hazard plan. The flaws in the investigation described in this paper result from simplifying assumptions but, as experience is gained from the comparison of actual movements to numerical predictions, it should be possible to design more accurate models of dolomitic subsidence.

The method is, potentially, a tool for use in calculating the cost of subsidence, in the locating of areas requiring additional investigations, and in designing monitoring systems.

At this stage, the technique described is more useful for the assessment of potential subsidence over an area, rather than at a specific structure.

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A COMPARISON OF EMPIRICAL AND DETERMINISTIC PREDICTION OF MINING SUBSIDENCE

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Abstract

All theories for predicting ground subsidence in mining areas are based either on empirical (statistical) models obtained through fitting of selected displacement functions into observed deformations or on deterministic modelling of the load-deformation relationship.The authors have been involved in a study of surface subsidence produced by extraction of a steeply inclined coal seam in difficult geological and topographical conditions of the Canadian Rocky Mountains.Actual displacements obtained from geodetic surveys and from telemetric tilt measurements have been compared with the predicted values obtained from empirical modelling using Budryk-Knothe's theory and from a deterministic model using the finite element method.The FEM approach,despite uncertainties in the geological input data,indicates obvious advantages over the empirical model.

Introduction

Since 1975 the Department of Surveying Engineering at the University of New Brunswick has been involved in deformation studies in geotechnical,mining and geophysical projects.The research programme has been concentrated on:

- development of new technologies for monitoring surveys,
- development of a generalized approach to the geometrical analysis of deformation surveys, and
- development of a methodology for predicting ground subsidence in diversified geological and topographical conditions of coal mining in Western Canada.

The research on the new technologies has led to a development of a telemetric system for a year-round and fully automatic monitoring of ground subsidence.The system was developed at a request of the Canada Centre for Mineral and Energy Technology to monitor mining subsidence in the difficult topographic and climate conditions of the Canadian Rocky Mountains (Fisekci and Chrzanowski,1981).The field stations of the telemetry system are battery operated and they have been designed to function in temperatures of -25°C without recharging the batteries for 9 months.The field stations are radio linked with a conveniently located master station which can be computer controlled from any point across the continent.Results of a three years' field test are given in (Chrzanowski et al.,1983a).

The research in the geometrical analysis of deformation surveys has led to a development of a generalized approach in which any number of observation epochs and any type of observables (geodetic and geotechnical measurements) can be analysed simultanously to derive deformation parameters for any type of deformations (including non-linear models with discontinuities). The approach is based on the least squares fitting of a selected deformation model into the survey data.The selection of the deformation model is based on a trend analysis,statistical testing of the model,and statistical significance of the calculated deformation parameters.Full details of the generalized approach are given in (Chen,1983) and an example of its application is in (Chrzanowski et al.,1983b).

The research in the prediction of mining subsidence started only three years ago .Preliminary results are summarized in this paper.

General strategy in the analysis of deformations

The following two aspects of deformations should be distinguished in the design and analysis of deformation surveys:

1) geometrical,if we are interested only in geometrical status of the deformable body,the change of its shape and dimensions,
2) physical,if we want to determine the state of internal stresses in the body and,generally,the load-deformation relationship.

In the first case,information on the acting forces and stresses and on physical properties of the body are of no interest to the interpreter or are not available.As a final result of the geometrical analysis of deformation surveys usually only relative displacements of discrete points are given with their variance-covariance matrix.The geometrical analysis is of a particular importance when the deformable structure is supposed to satisfy certain geometrical conditions such as verticality or alignment of some of its components.In that case,the results of the deformation surveys are directly utilized in adjustment of the geometrical status.

In a more refined geometrical analysis when an overall picture of the geometrical status is required the displacement field for the entire body is approximated through the least squares fitting of a selected deformation model into the observed displacements using,for instance,the aforementioned generalized approach.

In the case of the physical analysis of deformations,the load-deformation relationship may be modelled by using either an empirical (statistical) method through a correlation of observed deformations with the observed loads or a deterministic method which utilizes information on the loads,properties of the material,and physical laws governing stress-strain relationship.The empirical method is of an a-posteriori nature because it utilizes the past data through a regressive analysis in establishing a prediction model of deformations as a function of loads.A discussion on empirical modelling and its statistical testing has been given in another paper (Chen and Chrzanowski,1982).The deterministic method is of an a-priori (design) nature.It is usually based on approximate solutions using numerical analyses among which the finite element method has become a very powerful tool.Both methods,the empirical and deterministic complement each other.The deterministic model of the load-deformation relationship can be "calibrated" (enhanced) through a comparison with the empirical model.On the other hand,the design of the deformation surveys,which are used later on in empirical modelling should be designed on a basis of the deterministic model so that the location of survey points and type of instruments used would give the best possible information on the type and location of the maximum deformations.

Due to many uncertainties in the deterministic modelling of deformations the theoretically calculated displacements $\underline{d}_D$ (or any other deformation quantities) will generally depart from the observed values $\underline{d}_o$.The discrepancies may be due to the approximations of the deterministic method or due to:

-imperfect knowledge of the material properties,for example,errors in the elasticity constants,

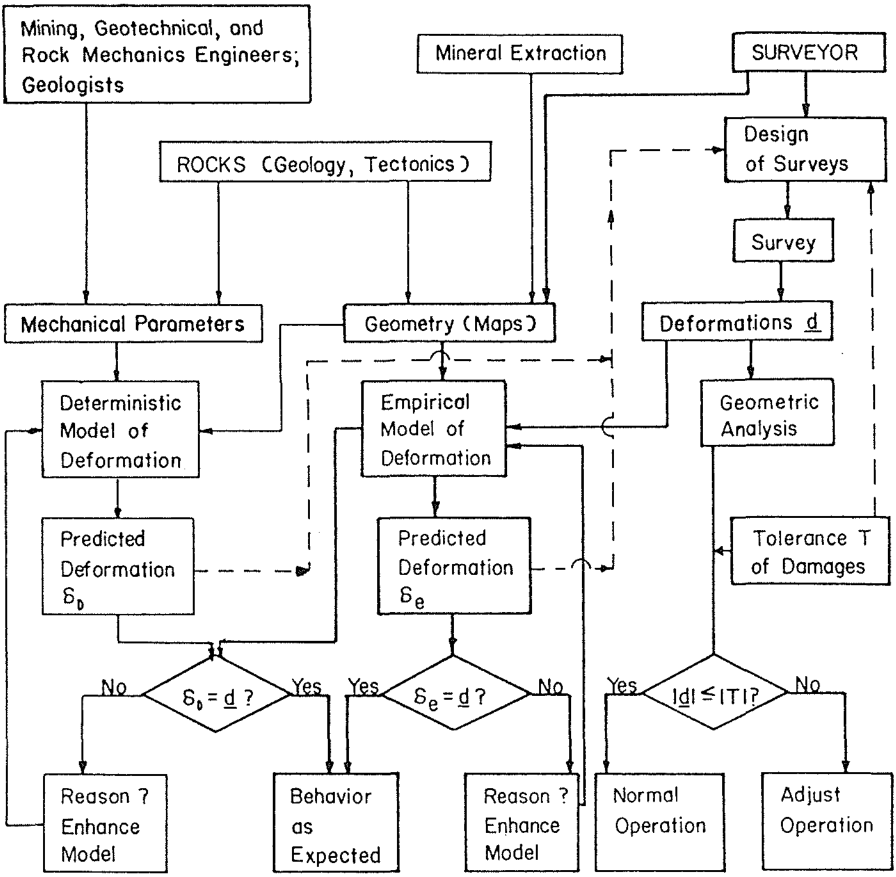

FIG.1. Flowchart of operations involved in the strata control and in the prediction of mining subsidence.

-wrong modelling of the behavior (elastic instead of plastic or creep neglected,etc.) of the material;

-measuring errors of the empirical method;

-measuring errors in sampling and incomplete sampling of loading effects. The investigation of the discrepancies is useful in gaining a better knowledge of the behavior of the deformable body.Statistical tests on the discrepancies may help in finding their nature and source.If the discrepancies are tested to be of a systematic nature then the deterministic and empirical methods are combined for the interpretation of the deformation measurements.One way of doing this is to assume that the systematic discrepancies are caused,for example,only by the improperly chosen material parameters, say E and ν .In this case,new "calibrated" values of the parameters are estimated by applying the least squares criterion:

$$\min_{E,\nu} \left\{(\underline{d}_D - d_o)^T C^{-1}(\underline{d}_D - d_o)\right\} \qquad (1)$$

where C is a variance-covariance matrix of the differences $(\underline{d}_D - \underline{d}_o)$. This method of the calibration of the constants of the deterministic model may lead to physically unacceptable values of the calibrated quantities if the real reason for the discrepancies is of a different nature. In such a case one has to try another approach.

Figure 1 (Chrzanowski and Hart, 1983) summarizes the interaction between the empirical and deterministic modelling for a purpose of controlling the strata movement in mining areas and for predicting the mining subsidence.

Description of the study area

The mining area under investigation is a coal mining operation in the Rocky Mountains in southern British Columbia. A 12 m thick coal seam, dipping at an average angle of 35°, is being extracted in panels of about 700x200m using a hydraulic method of extraction with roof caving (Fisekci et al., 1981). Figure 2 shows a planimetric map of one of the panels which has been used for the preliminary study of the mining subsidence in the area. Figure 3 shows a vertical crossection of the topography and the coal seam.

The extraction of the panel took place between July 1980 and August 1981. The mining company provided periodical monitoring surveys of the surface movements from the beginning of the extraction until the summer of 1983 when the ground subsidence became negligibly small. Figure 2 shows locations of the survey stations which were positioned with the electronic tacheometer AGA-700 from reference stations located below the outcrop of the coal seam. The results of the subsidence surveys for the total period 1980-1983 are summarized below.

Station No:	Δx (m)	Δy (m)	Δz (m)
1	-0.87	-0.10	-0.89
2	-0.95	-0.17	-0.91
4	-1.37	-0.47	-1.31
5	-1.41	-0.38	-1.73
A1	-1.42	-0.19	-1.60
A2	-1.61	-0.42	-1.77
A3	-1.66	-0.45	-1.95
A4	-1.44	-0.69	-1.55
A5	-0.77	-0.80	-0.38
B1	-1.51	-0.51	-1.76
B4	-1.96	-0.67	-2.18
T0	-1.23	-0.15	-1.08
T1	-1.12	-0.05	-0.81
T2	-1.76	-0.46	-1.57
T3	-1.72	-0.46	-1.42
T4	-1.07	-0.25	-0.99

Due to difficult winter conditions (up to 5m of snow) the geodetic surveys were limited to the short summer season. In order to provide the time related subsidence information, the aforementioned telemetry system for continuous monitoring of ground movements has been developed and tested in the area using bi-axial tiltmeters (Chrzanowski et al., 1983) as the sensors. Locations of the tiltmeter stations are shown in figure 2. Total tilt components over the period of 1980-1983 are listed below. Stations T0 and T2 are not listed because their operation was disrupted by wild animals.

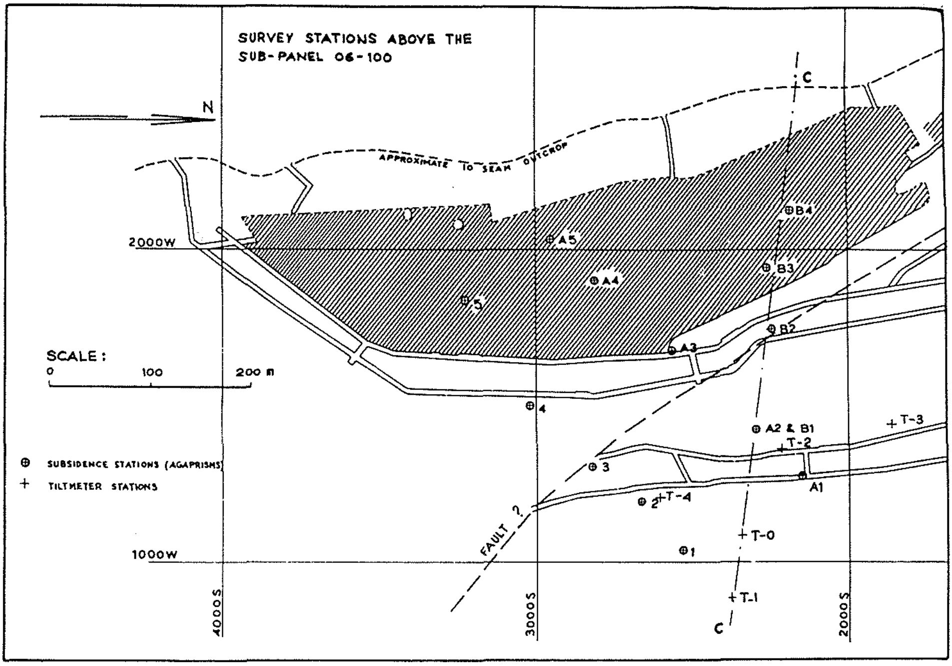

FIG.2. Map of the extracted panel and locations of geodetic and telemetric monitoring stations.

Station No:	Tilt in -x direction (mm/m)	Tilt in +y direction (mm/m)
T1	-9.4	-2.3
T3	-12.8	+6.8
T4	- 8.8	-5.1

Aerial photogrammetry has also been utilized in the area as a part of the research programme (Faig and Armenakis,1982).However,the results have not yet been included in the comparison of the monitoring surveys with the empirical and deterministic models of the deformation.

The extraction of the panel,besides the listed displacements of the survey points,produced surface cavings above the upper edge of the panel and long south-north cracks near the mountain ridge.Due to a very limited knowledge on the geology and tectonics of the area the cracks could not be readily explained.The geological fault (Fig.2) which was approximately mapped at the level of the mining workings,could be a possible explanation. However,the geometry of the fault,its dip angle,and depth have not been identified.Therefore,a comparison of the actual subsidence with the predicted values for those uncommon topographical and mining conditions has been of a great interest.

Empirical model of the subsidence

Most of the existing empirical methods have been developed in Central Europe where a vast amount of survey data is available.A good review of the empirical methods is given by Kratzsch (1983).In North America,in general,monito-

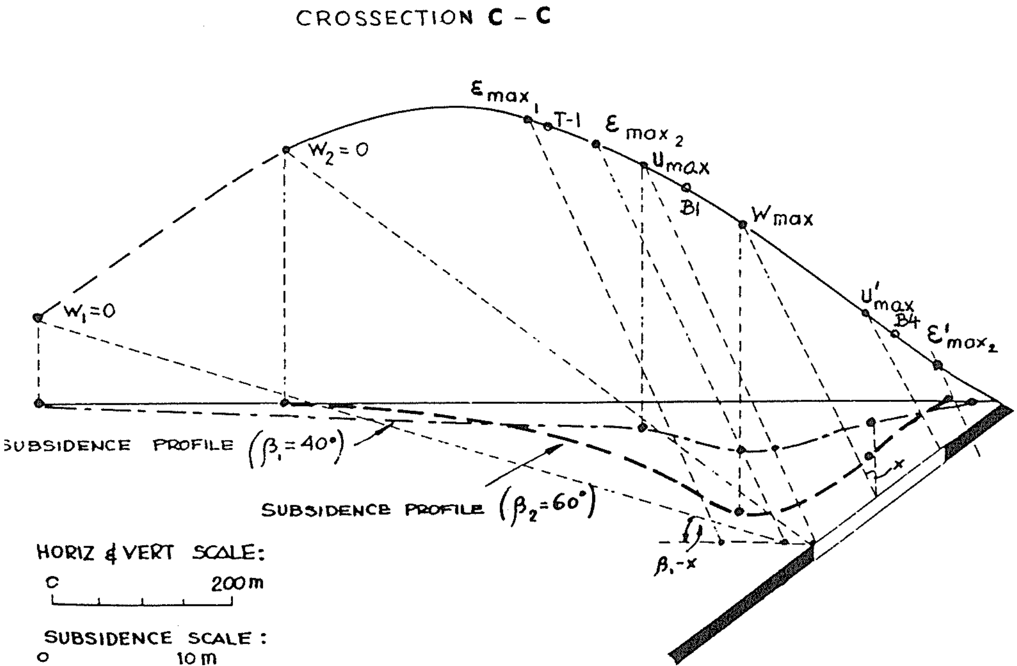

FIG.3. Vertical crossection of the study area and predicted subsidence profiles using the Budryk-Knothe's theory.

ring of mining subsidence is still at a pioneering stage and the observation data is insufficient for developing own empirical models for individual mining districts.Therefore,the authors adapted one of the European prediction methods to the study area.

Most of the empirical theories dealing with the prediction of the mining subsidence are based on empirical functions of influence derived from a large number of monitoring surveys .The functions describe a relationship between certain empirical parameters and the expected subsidence at selected points. Examples of frequently used parameters are:limiting angle β of influence(angle from the horizontal at the edge of the extracted area to a point on the surface where the subsidence is negligibly small),critical radius r of the extracted area which produces the maximum subsidence,bulking coefficient of the caved-in rocks in the extracted area,etc.

The well known Budryk-Knothe's theory (Budryk and Knothe,1953 and Knothe, 1953) is one of the easiest to apply in practice and,therefore,it has been selected by the authors to predict the subsidence in the study area.The theory was developed for the coal mining district in Upper Silesia in Poland.

The theory is based on the influence function,which for an extracted area A in an x,y coordinate system gives the subsidence W at a point (x,y) on the surface in the form:

$$W(x,y)=(W_{max}/ r^2) \iint_A \exp\left[- (x^2+ y^2)/r^2\right] d_A \qquad (2)$$

where W_{max} = a.g for the extraction area equal to or larger than the critical area where g is the thickness of the extracted mineral and 'a' is the empirical bulking coefficient which for the extraction with the roof caving is taken between 0.75 to 0.90.The critical radius $r = H.\cot\beta$ where H is depth of the extraction,and β is the empirical limiting angle to a point at which the final subsidence may reach 1.22% of W_{max} if the extracted area A is equal to or larger than the critical area.

For the Polish coal district the angle β may vary from 35° to over 60°. Since no information on the angle β in the study area was available,the authors performed their analysis for two values: $\beta_1 = 40^{\circ}$ and $\beta_2 = 60^{\circ}$.The B-K theory applies to horizontal or slightly inclined (up to 15°) coal seams In the case of the steeply inclined deposit in the study area the method had to be modified by the authors by shifting all the subsidence parameters (points of W_{max},limiting angles,points of maximum strain,etc.)by an empirical value of 0.7α (Kratzsch,1983),where α is the dip angle.The Budryk-Knothe's theory has been applied in a graphical mode (a circle of radius r drawn on tracing paper at the scale of the mining map and divided into segments of equal influence) as explained in (Chrzanowski et al.,1980).

Figure 3 summarizes the results of the analysis in the study area for the two alternative limiting angles $\beta_1 = 40^{\circ}$ and $\beta_2 = 60^{\circ}$,average depth H=200 m dip angle $\alpha = 35^{\circ}$, g=12 m, and a = 0.75.The results predict that the maximum subsidence could reach the value of 3.15 m and 7.50 m for β_1 and β_2 respectively.The subsidence influence could reach the surface far beyond the ridge of the mountain where,unfortunately,no monitoring surveys were conducted in order to confirm the value of the limiting angle.

Maximum values of other deformation parameters were determined from the following relationships (for β_1 and β_2 respectively):

maximum tilt: $T_{max} = W_{max}/H\cot\beta$ = 13 mm/m and 65 mm/m,
maximum strain: $\varepsilon_{max} = 0.6T_{max}$ = 7.8 mm/m and 39 mm/m,
maximum horizontal displacements: $U_{max} = 0.4\ W_{max}$ = 1.26 m and 3.00 m.

The values of W_{max} and T_{max} for $\beta = 40^{\circ}$ agree very well with the observed quantities when considering that the Budryk-Knothe's theory has been applied in its crudest form.Also the location of the surface fractures on the ridge agree quite well with the zone of ε_{max1}.However,in order to apply the theory for predicting the influence of the future extraction of neighbouring panels one should determine the actual value of the limiting angle for the area and the coefficient a.One should note that the suspected fault has not been considered in the empirical prediction.Therefore,the above agreement may be accidental.

Deterministic model of the subsidence

The finite element method has been applied in the deterministic modelling of the subsidence.Computer programmes for 2-D and 3-D finite element analyses have been developed by the co-author (Szostak-Chrzanowski,unpublished) for the purpose of the study.The 2-D analysis utilizes quadrilateral elements with a six point integration technique in order to increase the accuracy of the FEM solution.

Due to very limited and uncertain geological information in the study area,only a 2-D analysis in the simplest possible form was applied to the crossection C-C.According to the available data,the strata above the extracted panel consists of a medium strong sandstone and shale formation with the aforementioned suspected fault.Figures 4 and 5 show the mesh of the ele-

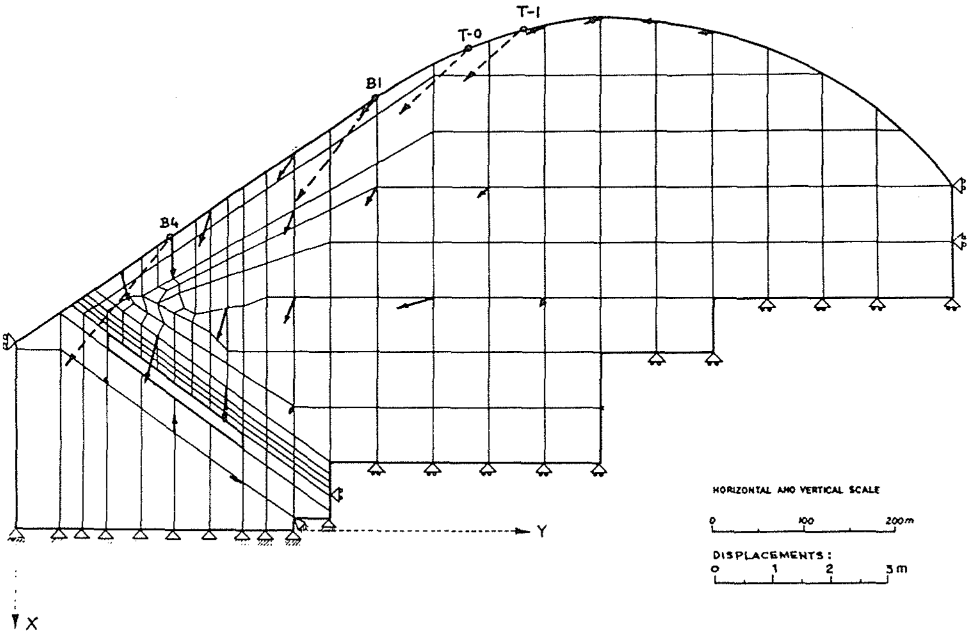

FIG.4. A comparison of observed and FEM displacements after introducing the 'no tension' elements (marked +) above the extracted area.

ments and the boundary conditions.The weight of the rocks has been introduced as body forces in each elemnt.The simplest approach based on the "gravity turn-on" (Kulhavy,1974) and "no tension elements" methods has been applied in the preliminary analysis in which the displacements,strains and stresses caused by the mining operation were calculated iteratively as differences between the initial (no excavation,homogenous and elastic material) and consecutive solutions.Initially,the values of the module of elasticity and the Poisson ratio were accpeted to be the same for all elements: $E = 10^6 kN/m^2$ and $\nu = 0.3$ with the specific weight of the strata material $\gamma = 27 kN/m^3$. Later on,the value of E for those elements which have shown large extensional changes in the principal stresses was decreased to $E = 10^3 kN/m^2$.

Figure 4 shows sample displacements of nodal points after a third iteration and the observed displacements of the few survey stations.The disagreement is very large.Also the calculated strains (about 0.1 mm/m)near the ridge of the mountain could not encounter for the fractures of the surface.Therefore,in the next solution the suspected fault has been introduced into the model by decreasing the module of elasticity in the marked elements to $E = 10^3 kN/m^2$.This solution gives a good agreement with the observed displacements.Perhaps,even a better agreement could be reached by enhancing the FEM model (changing the mesh of the elements near the surface and in the fault zone,changing E and ν values) and using a more sophisticated approach,for example,the "stress transfer" method (Zienkiewicz et al.,1968) or some newer methods.However,any further enhancement of the model would be justified only in the case when more geological information would become available, because the FEM solution is only as good as good is the input data.

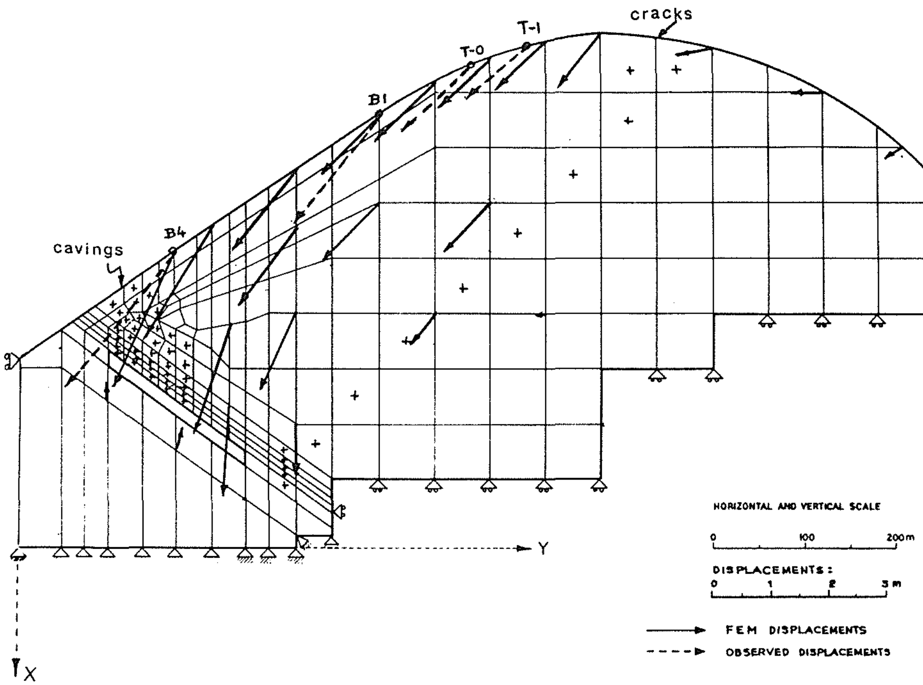

FIG.5. Sample FEM displacements after introducing the 'no tension' elements in the zone of the suspected fault.

Conclusions

Both,the empirical and deterministic models of the subsidence in the study area,gave reasonably good agreements with the observed data.In both cases, in order to obtain the agreement,certain assumptions had to be made. In the FEM model the best solution was obtained when the unconfirmed fault was introduced to the model.In the empirical approach,the value of the limiting angle had to be assumed and the agreement was obtained without the fault line which,apparently,would tremendously complicate the application of the Budryk-Knothe's theory.Both,the existence of the fault and the value of the limiting angle must be confirmed before any final conclusions on the comparison of two approaches could be made.However,if a need would arise to predict the future subsidence in the study area due to to the extraction of neighbouring panels,the authors would feel much more confident by using the FEM approach,even with the present geological uncertainties,rather than using the empirical model adapted from another mining region with the unconfirmed parameters.The detrministic method,besides that it gives much more information on the behavior of the whole deformable object than the empirical method,can be applied in any situation and in any conditions as long as the conditions are,at least,approximately known.The empirical methods can be applied only in the very similar conditions in which the empirical parameters have been determined.

Acknowledgements

This preliminary study was sponsored by the Canada Centre for Mineral and Energy Technology and by the Natural Sciences and Engineering Research Council of Canada.This paper was prepared during the authors' visit to the Swiss Federal Institute of Technology in Lausanne with a permission by prof.A.Miserez and with a help by Mr.J.R.Gros.

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MECHANICS AND ONE-DIMENSIONAL SIMULATION OF LAND SUBSIDENCE DURING THE PERIOD OF HEAD RECOVERY IN AQUIFERS

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Abstract

During the period of hydraulic head recovery within the confined aquifers of the multi-layered aquifer-aquitard system due to the control works against groundwater withdrawals in the Niigata plain in Japan, land subsidence and delayed uplift of the ground surface have been observed since about 1965. A discussion of mechanism of such phenomena is presented by considering the behaviour of groundwater flow within an aquitard. One-dimensional compaction and expansion of such system was simulated at two different sites. Each of the systems consisted of three wells. The observed data of transient aquifer-heads were used as known quantities in the calculations. Different storage coefficients for compaction and expansion and hydraulic conductivities for the aquitards are determined by trial-and-error. A set of different values of the ground parameters are taken in each of the three different depth levels. Simulation results agree with the observed data with reasonable accuracy over a decade.

Introduction

Since about 1835, land subsidence in the Niigata plain has been caused by extensive groundwater withdrawals from the confined aquifers with natural gas dissolved in water. According to the several controls against groundwater withdrawals since 1959, the hydraulic heads in aquifers have been increasing. In particular, since the control with water injection into gas reservoirs in 1973, the considerable head rise in the observation wells could be observed. However, in spite of these head increases in the aquifers, subsequent compaction and delayed expansion of the system was observed. See FIG. 1 and 2.

Mechanism (Yoshida, 1982)

For simplisity of explanation, we consider a simple system with a single aquitard sandwiched in between aquifers (FIG. 3). The hydraulic head h can be defined by

$$h = p_w / \gamma_w - z$$

where p is the pore water pressure, γ_w the unit weight of water and z the vertical coordinate taken to be positive downward. The consolidation process of an aquitard (the clay layer) is usually governed by the following basic equation:

$$\gamma_w m_v \frac{\partial h}{\partial t} = \frac{\partial}{\partial z}\left(K \frac{\partial h}{\partial z}\right) \quad \text{or} \quad S_s \frac{\partial h}{\partial t} = \frac{\partial}{\partial z}\left(k \frac{\partial h}{\partial z}\right) \qquad (1)$$

where m_v is the coefficient of volume compressibility, k the hydraulic conductivity, S_s the specific storage coefficient ($S_s = \gamma_w m_v$), and t the time.

When k is assumed to be constant for an approximation, Eq. (1) reduces to

$$\frac{\partial h}{\partial t} = c_v \frac{\partial^2 h}{\partial z^2} \quad \text{or} \quad S_s \frac{\partial h}{\partial t} = k \frac{\partial^2 h}{\partial z^2} \qquad (2)$$

where $c_v (= k/(\gamma_w m_v) = k/S_s)$ is the coefficient of consolidation.

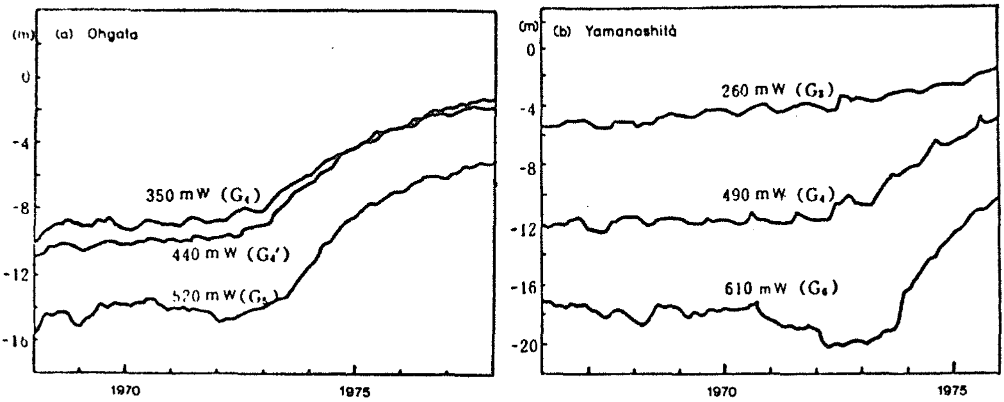

FIG.1 Transient hydraulic head in observation wells

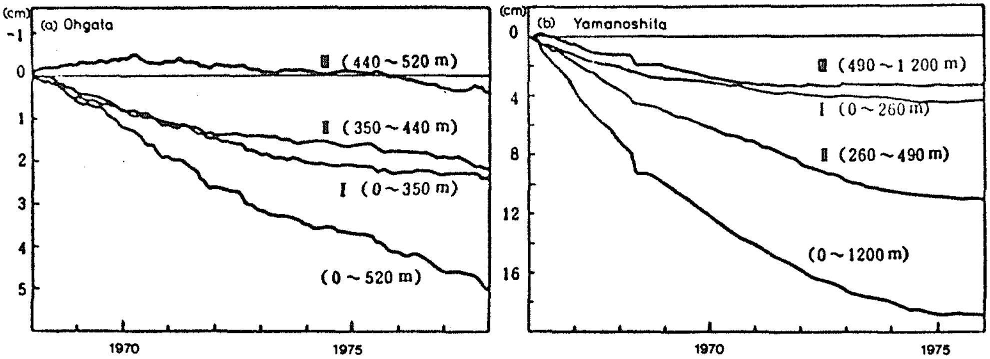

FIG.2 Transient cumalative compaction of each of depth intervals due to observation wells.

In using Eq. (2), the value of m_v or S_s is adopted to be different for compaction and for expansion of an aquitard as in a manner described by Helm (1972): namely, comparing the value of h at any z with h_{min} (= the minimum value of h in the past at the same point z), S_s or m_v is used as follows:

$$S_s = \begin{cases} S_{sv} = \gamma_w m_{vv} \text{ for } h \leqq h_{min} \\ S_{se} = \gamma_w m_{ve} \text{ for } h > h_{min} \end{cases} \qquad (3)$$

Similarily,

$$c_v = \begin{cases} c_{vv} & \text{for } h \leqq h_{min} \\ c_{ve} & \text{for } h > h_{min} \end{cases}$$

For simplisity in FIG. 3, it is supposed that after the heads in the aquifers decreased to the lowest level (B-B) from the static or equilibrium state (A-A) due to groundwater withdrawals, these heads are continuously rising to be (C-C) due to restriction against groundwater withdrawals. Under these conditions, the head within the aquitard may change from (A-A) to dotted curve(B-B'-B) and then to solid curve (C-C'-C). The system compaction as a whole may occur by mechanics as follows.

As the heads in the aquifers increase with time, the heads in the two outside layers within the aquitard (FIG. 3) increase almost simultaneously because $c_{ve} > c_{vv}$ for the coefficients of consolidation and because these outside layers are thin (but increase in thickness with time).

These outside layers within the aquitard show expansion and the layer in between the outside layers shows compaction simultaneously. The aquitard as a whole performs expansion until a certain period. The cause of this deformation behaviour of aquitard can be explained as follows:

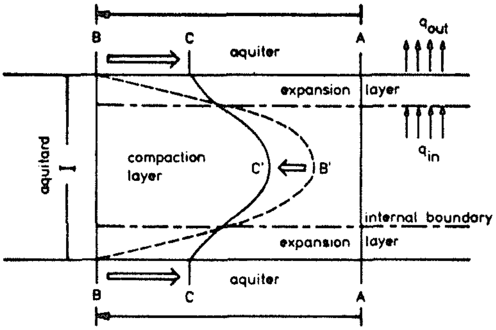

FIG.3 Schematic diagram showing transient head within an aquitard due to head change within aquifers.

(1) The water flux q_{in} out of the medial layer into the outside layer through a transient internal boundary within the aquitard is higher than the water flux q_{out} from the outside layer into the adjacent aquifer through the aquifer-aquitard interface: ($q_{in} > q_{out}$). This relation $q_{in} > q_{out}$ is reasonable related to the measurements of the gradients of hydraulic head both at the internal boundary and at the aquifer-aquitard interface respectively. Hence, the outside layers continue to expand due to the net flux $q_{in} - q_{out}$ ($> o$)

(2) On the other hand, the medial layer within the aquitard continues to compact because the descending residual head within

the medial layer is still higher than the ascending heads in the outside layers and so the water flux from the medial layer into the outside layers is subsequent.

(3) The aquitard shows compaction as a whole corresponding to the quantity of water (q_{out}) squeezed from the aquitard into the adjacent aquifer. This is independent of the water flux q_{in} within the aquitard.

(4) As a result of both the subsequent rising head in the aquifers and the increase in thickness of the outside layers, the water flow from the aquifer into the aquitard occurs due to the converse gradient of the hydraulic head at the aquifer-aquitard interface. Therefore, the aquitard begins to expand as a whole according to a net gain in water flux from the adjacent aquifers into the aquitard.

The above mentioned is the reason for the occurence of the subsequent compaction and the delayed expansion of the aquitard over the period of the transient head recovery in the aquifers. The mechanism mentioned is not contradictory to applying S_{sv} or S_{se} for compaction or expansion respectively and in addition can explain that the same phenomenon occurs to a less degree even when $S_{sv} = S_{se}$.

Next, computed examples for the same simple system as shown in FIG. 3 are presented. According to FIG. 5 the hydraulic head of aquifers is assumed to decrease by 5 m for 1800 days (at the rate of nearly 1.0 m/yr) and then to increase at the same rate, as shown with dot-dashed line. On this condition, transient hydraulic head distribution within the aquitard of depth intervals of 10 m is calculated by Eq. (2) (FIG. 4) and cumalative settlement (compaction) is obtained by the water flux from the aquitard to the aquifers (FIG. 5).

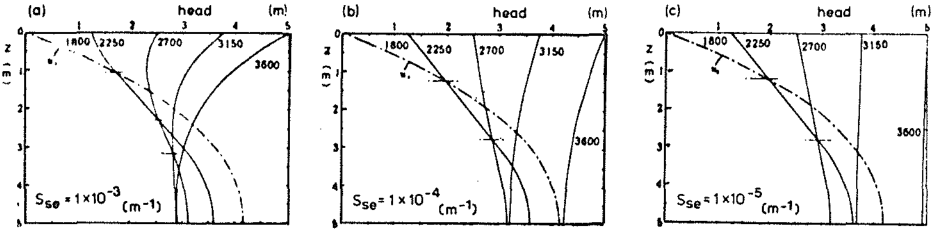

FIG. 4 Transient head distribution within an aquitard due to head changes within aquifers

In these computations, the adopted values from k and S_{sv} are 0.5×10^{-8} cm/sec $= 1.58 \times 10^{-3}$ m/yr and 1×10 m^{-1} respectively and the values for S_{se} and hence for S_{se}/S_{sv} are 1×10^{-3} m^{-1} and 1 for case (a), 1×10^{-4} m^{-1} and 1/10 for case (b) and 1×10^{-5} m^{-1} and 1/100 for case (c) respectively. FIG. 5 demonstrates that the smaller the ratio S_{se}/S_{sv} is the more delayed expansion occurs and the less expansion amounts.

In FIG. 4 case (a), $S_{se}/S_{sv} = 1$, the distribution of h within the aquitard becomes concave near the aquifer-aquitard interface. This state reaches at a comparatively early period (after

2700 days) and the aquitard already shows expansion as a whole at the same time. As S_{se}/S_{sv} is, however, smaller as in case (b) and case (c), hydraulic head or pore water pressure in the outside expansion layer within the aquitard rapidly follows the head rise in the aquifers (FIG. 4 (b) and (c)). This is enabled because $c_{ve} = k/S_{se}$ is larger than $c_{vv} = k/S_{sv}$ and because the thickness of the outside expansion layer δH is much smaller than H at the early stage and consequently because $S_{se}(\delta H)^2/k \ll S_{sv}(H/2)^2/k$ in their time constants (where H is the thickness of the aquitard and δH increases from zero with time). Namely, when $c_{ve} = k/S_{se}$ is larger, in an outside layer a concave distribution of h does not occur easily and consequently compaction continues some time after the beginning of head rise in the aquifers and so expansion is delayed. Let us suppose the rate of head rise in aquifers as $v = dh/dt$. This delayed time is considered to depend on a parameter c_{ve}/v.

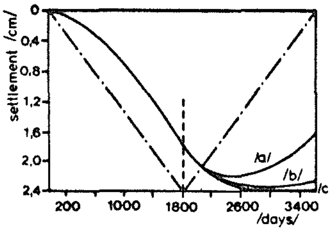

FIG.5 Transient cumalative settlement of aquitard (solid lines) due to head change within aquifers (dot-dashed line).

Simulation (Yoshida, 1983)
The geological models at two sites (Ohgata 520 meters and Yamanoshita 1200 meters) in the Niigata plain were determined as shown in FIG. 6 (a) and (b) respectively, referring to boring data.

In order to calculate the initial distributions of residual excess pore water pressure head within the aquifers in 1968 for Ohgata and in 1966 for Yamanoshita, in each aquifer with an observed well the history of transient head from 1949 to the initial time are assumed to be approximately as in FIG. 7. This refers to other field data. The heads in all aquifers above aquifer (I) with the shortest observed well (FIG. 8) is assumed to be equal to the values of head of the aquifer (I). The heads in the aquifers both between aquifer (I) and aquifer (II) with the second longer observed well and between aquifer (II) and aquifer (III) with the longest observed well are assumed to be linear to depth through the values of head of these aquifers (I), (II) and (III). A set of estimated values for k, S_{sv} (or m_{vv}) and S_{se} (or m_{ve}) of aquitards are by trial-and-error determined to take different values in each of three depth levels at each site (Table 1), so that computed compactions fit closely with the observed compactions.

For an example of calculated results, the initial distribution of residual excess pore water pressure in 1968 and the distribution of head in 1974 for Ohgata are demonstrated in FIG. 9. This figure shows that excess pore water pressure within aquitards diffuses relatively rapidly outward (like C_4 in FIG. 9), whereas excess pore water pressure remains almost constant in the case of wide aquitard intervals (like C_1, C_3 and C_6 in FIG. 9).

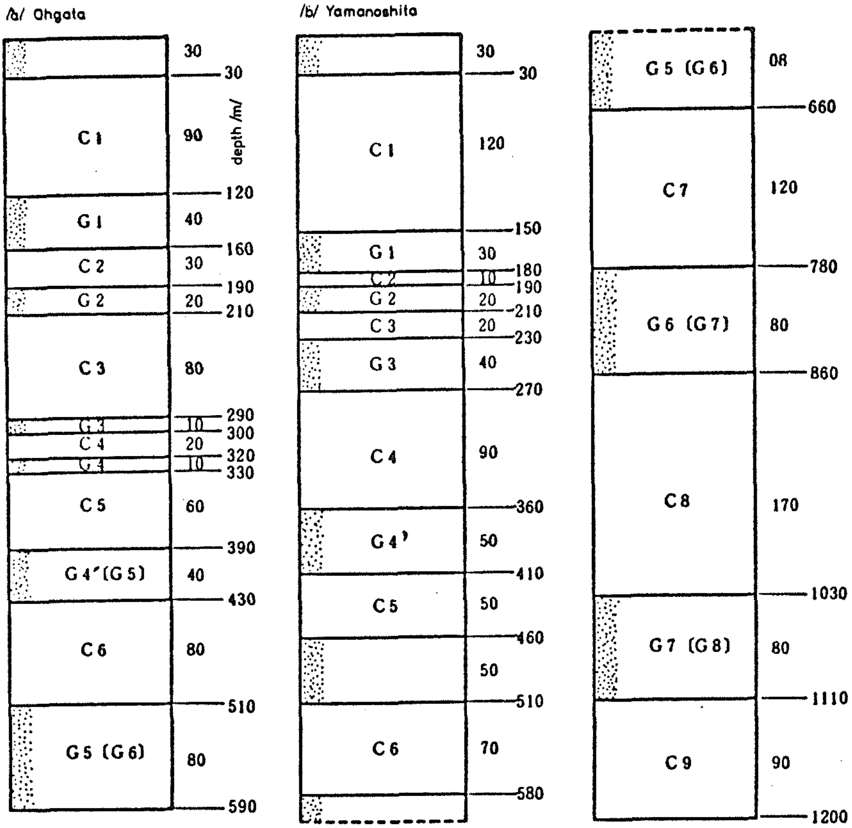

FIG.6 Geological model for aquifer-aquitard system (C:aquitard, G:aquifers)

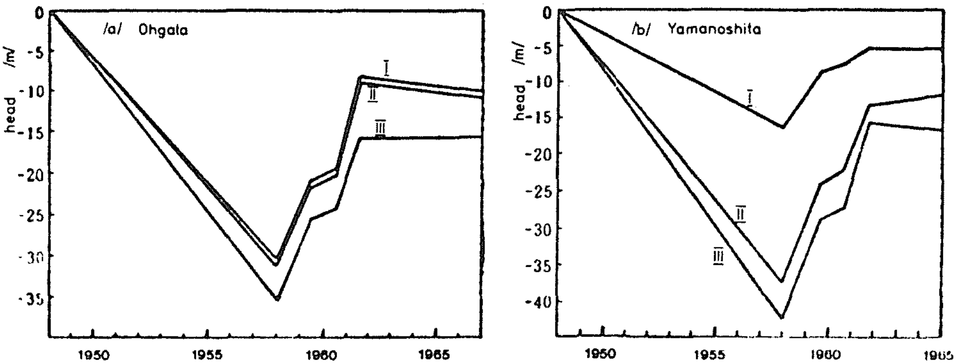

FIG.7 Transient head changes assumed for aquifers.

Table 1 Values of ground parameters for aquitards in each of three depth intervals.

Site	Depth (m)	k (cm/sec)	S_{sv} (m^{-1})	S_{se} (m^{-1})
Ohogata	I (0 ~350)	3.0×10^{-9}	6.0×10^{-4}	6.0×10^{-6}
	II (350 ~ 440)	6.0×10^{-9}	1.3×10^{-3}	1.3×10^{-5}
	III (440 ~ 520)	9.0×10^{-10}	1.8×10^{-4}	1.8×10^{-6}
Yamanoshita	I (0 ~ 260)	3.0×10^{-8}	5.2×10^{-3}	5.2×10^{-5}
	II (260 ~ 490)	2.0×10^{-8}	2.2×10^{-3}	1.1×10^{-4}
	III (490~1200)	1.0×10^{-9}	5.0×10^{-4}	7.0×10^{-5}

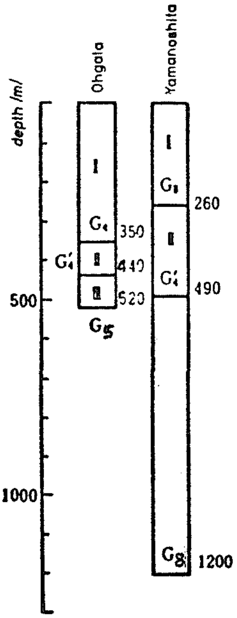

FIG.8 Depth of observation wells and three depth intervals distinguished by the observation wells.

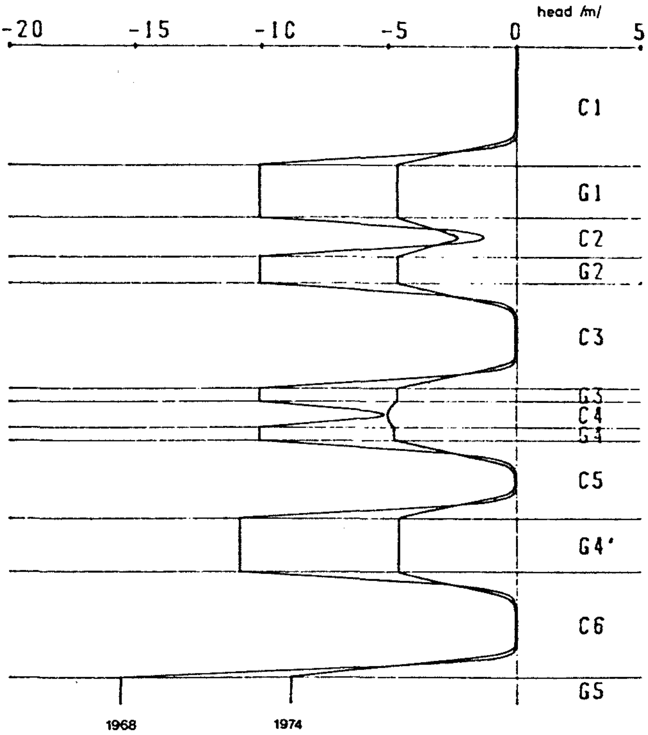

FIG.9 Distributions of residual excess pore water pressure head calculated for Ohgata.

Before the complete recovery of the head within the aquifer, land subsidence continues because the residual pore water pressure within the aquitards, especially the thick ones, remains higher than the ascending head within the adjacent aquifers. It is to be noted that in 1974 the head distribution within a thin layer like C_4 appears to be concave. This proves for itself an expansion layer in this year.

The results of simulation for transient cumulative compaction of each of the three different depth intervals (a) from 1968 to 1978 for Ohgata and (b) from 1966 to 1976 for Yamanoshita are shown in FIG. 10 (a) and (b).

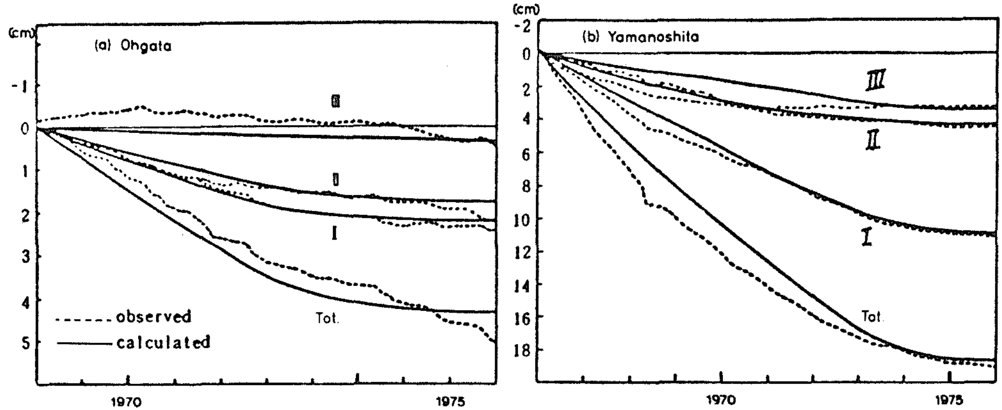

FIG.10 Transient cumalative compaction and expansion of each of depth intervals observed (dotted lines) and computed (solid lines).

These figures show that the computed compaction agree within the range of a few millimeters with the observed data of the three observation wells, which are different in depth at each field site.

Consideration of expansion of aquifers

The storage coefficient S'_{sv} for compaction of aquifers is far less than S_{sv} of aquitards, whereas the storage coefficient S'_{se} for expansion of aquifers in comparatively close to the storage coefficient S_{se} of aquitards. Therefore, in uplift of ground surface due to head rise in aquifers, aquifer expansion may not be ignored. In order to confirm the effect of expansion of aquifers on the total expansion of a system, simulations considering expansion of aquifers for Yamanoshita was carried out. For simplisity the storage coefficient S'_{se} for expansion is taken to be 1.0×10^{-5} (m^{-1}) for all aquifers. First, a simulation result by using the same values of ground parameters for aquitards like in Table 1 is shown in FIG. 11 (a). This diagram shows that measurement of aquifer expansion cannot be ignored if the assumed value 1.0×10^{-5}(m^{-1}) of S'_{se} is not too large. Next, with the fixed value of S'_{se} like above, the other ground parameters are trial-and-error redetermined as in table 2 so that computed compaction and expansion fits closely to the observed data (FIG. 11 (b)). This agreement in FIG. 11 (b) becomes a little better than the result of FIG. 10 (b) by not considering aquifer expansion.

In future, more reasonable estimations and the uniqueness of a set of values of ground parameters have to be discussed.

Table 2 Values of ground parameters for aquitards in each of three depth intervals used in FIG.11(b).

Site	Depth (m)	k (cm/sec)	S_{sv} (m^{-1})	S_{se} (m^{-1})
Yamanoshita	I (0 ~ 260)	3.0×10^{-8}	5.5×10^{-3}	5.5×10^{-5}
	II (260 ~ 490)	2.3×10^{-8}	2.3×10^{-3}	1.2×10^{-4}
	III (490~1200)	7.0×10^{-9}	1.3×10^{-4}	1.3×10^{-6}

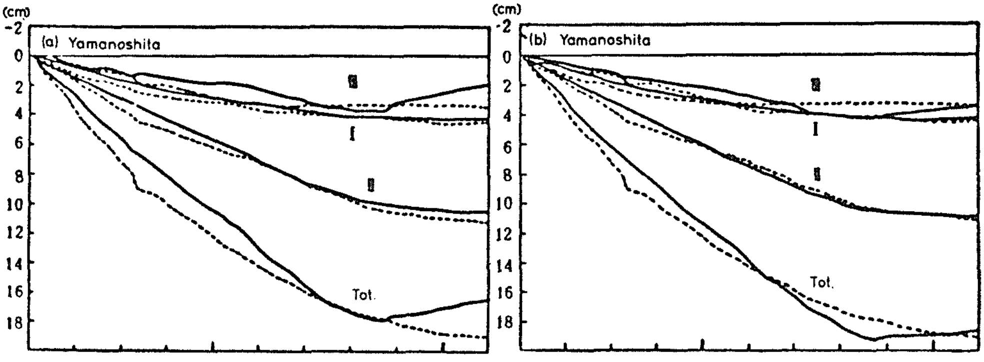

FIG.11 Transient cumalative compaction and expantion of each of depth intervals calculated by considering expansion of aquifers (solid lines).

Acknowledgment The authors are much indepted to Professor S. Yamamoto of University of Rissho, Japan and to Dr. Czurda of Innsbruck University, Austria for their valuable suggestion and advice.

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A MATHEMATICAL MODEL FOR THE HEAD VARIATION OF THE AQUIFER AND THE PREDICTION OF LAND SUBSIDENCE

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Abstract

The prediction for the head variation of the confined aquifer and the maximum possible quantity of development under certain head-variation conditions are the main topic of the control of land subsidence.

There are measured data on groundwater development,the head of the aquifer and the deformations of the layers in different depths,which have been continuous for many years in Shanghai area.With these measured really data and the digital time-sequence analysis method,the ARMA model for the head variation can be determined.A similar mathematical model can also be deduced from the theory of the groundwater steady-flow motion.

Further,the source function,the basic solution of the equation of the groundwater non-steady flow motion,can be used and simplified to obtain the mathematical model for the head variation and the result of try for the calculation proves to be reasonably acceptable.With this model,a reasonable layout of developping wells can also be made.

The parameters of the model are discussed in this paper and the results of the calculation by the method mentioned above are also given.

1.Relationship Between the Quantity of Development of Groundwater,the Head of the Confined Aquifer and the Deformation of the Soil Layer

The city of Shanghai is situated on the alluvial plain of littoral lithofacies near the estuary of the Yantze River, and the groundwater is developed from the confined aquifers between the

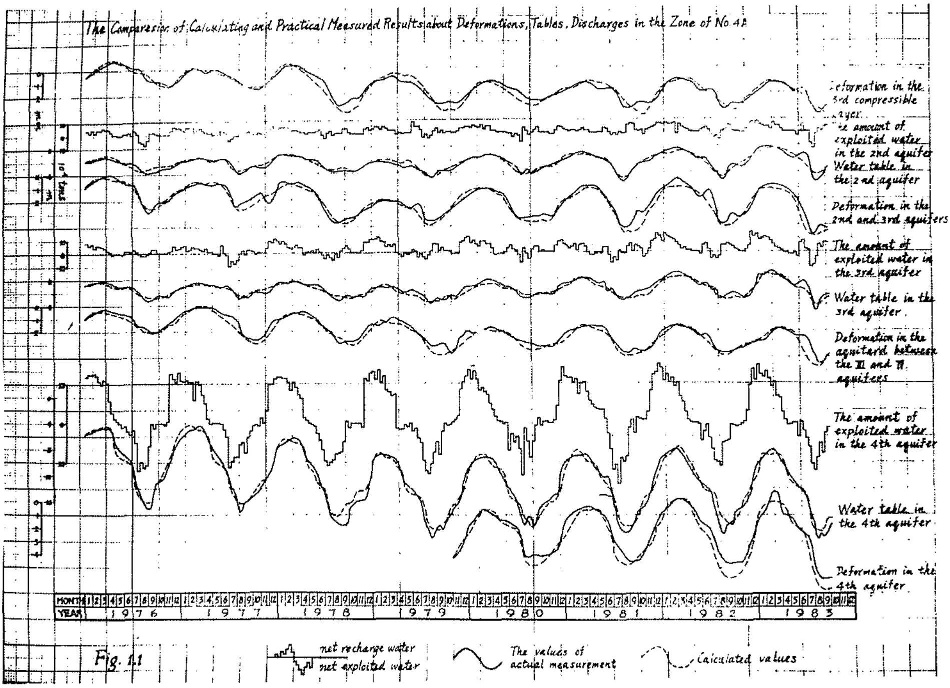


Fig.1.1

depths of 70m to 250m.Long time a large quantity of groundwater had been developed in the past (the maximum development was 550,000 tons per day),which caused the head of the confined aquifer to drop heavily, yet it did not damage the bearing-pressure of the aquifer.At present,the daily maximum quantity of development of water is just below 80,000 tons.With the injection of water into the aquifer in winter (to store up cold source so as to increase the efficiency of groundwater in use in summer), together with the recharge of water and the elastic release of water in the aquifer, the head of the aquifer shows a periodic up-and-down variation every year.The rise and fall of the head cause the expansion,rebound,consolidation and compaction of the layer.The relationship between three is obvious,as shown in Fig.1.1.

2.The Average Mathematical Model ARMA(1,1) for the Natural Regressive Slip of the Head Variation

It can be seen from Fig.1.1 that the development of groundwater and the head variations show a steady periodic regulation every year,and therefore a linear steady model can be formed by the digital time-sequence analysis.Consequently,ARMA(p,q) model is used for simulation,when taking it into consideration that the main factors of disturbance in the head variations of the aquifer are the development or injection of groundwater,therefore adopting variaus models formed by (p,q) were simulated and error analysis was made on the posterior prediction.Finally,the ARMA(1,1) model for the head variations of the aquifer is determined as follows:

$$H_{t+1}=a_0+a_1Q_{t+1}+a_2H_t \qquad (2.1)$$

where H_t is the head level of the confined aquifer at t,in m;

Q_t is the quantity of groundwater between t-1 and t, in 10,000 tons;

t is time base, taking 10 days as a unit.

3.The Mathematical Model for Head Variation Deduced from Groundwater Steady Flow Theory

The development of groundwater causes the head to drop,and assuming that the aquifer is isotropic and the head drop is like the shape of the cone of depression,the boundary of recharge of groundwater is similar to a circle.

See right Fig.3.1,the head at the center of the cone of depression at t is H_t,and the boundary head at the radius of influence R is H_0.R and H_0 are constants according to the steady flow theory.The essential factors that affect the variation of H_t are the quantity of development Q_{t+1} during the time interval between t and t+1,and the quantity of recharge q_{t+1}.If the head at the center of the cone of depression becomes H_{t+1} at t+1, the following can be deduced from the steady flow theory:

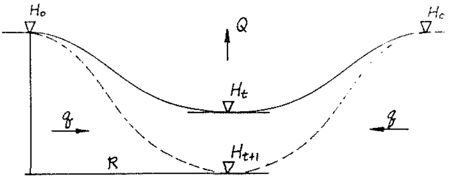


Fig. 3.1 The schematic diagram of depression of water table

$$H_{t+1}-H_t=\frac{KW}{R}(Q_{t+1}-q_{t+1}) \qquad (3.1)$$

The hydraulic gradient at t is: $$J=\frac{H_0-H_t}{R} \qquad (3.2)$$

According to Darcy's law: $$\frac{dq}{dt}=KW(\frac{H_0-H_t}{R}) \qquad (3.3)$$

We can find q_{t+1} by integrating equation(3.3),and after substituting it into equation(3.1) and rearranging,we have

$$(1-\frac{K^2W^2}{2\ R^2})H_{t+1}=-(\frac{K^2W^2}{R})H_o+\frac{KW}{R}Q_{t+1}+(1+\frac{K^2W^2}{2\ R^2})H_t \quad (3.4)$$

where K is the average coefficient of permeability of the aquifer within the area of the cone of depression,in m/day;

W is the area of the cross-section through which water passes the boundary of recharge, $W=2\pi R^2M$, in m^2;

M is the thickness of the aquifer, in m.

Let $a_o=\frac{2\ K^2W^2}{K^2W^2-2R^2}H_o$ (a)

$a_1=\frac{2RKW}{2R^2-K^2W^2}$ (b)

$a_2=\frac{2R^2+K^2W^2}{2R^2-K^2W^2}$ (c)

Then the equation(3.4) can be simplified to $H_{t+1}=a_0+a_1Q_{t+1}+a_2H_t$, which is identical with the equation(2.1) in form.The meanings of H,Q and t are also the same.

4.The Research of the Stability of the Parameters of the Model and Their Laws

Every calculating year (from the first ten days in Oct. to the last ten days in Sept. next year)is divided into two parts:head-rising period and head-falling period,according to the consumption of water and head variations in Shanghai,and the parameters of the model have been found for the head-rising and head-falling periods respectively by the Least Square Method.

The parameters given in Tab.4.1 were found conversely by substituting into Eq.(2.1) the head values of the IV confined aquifer near Labour Park,Shanghai and the consumptions of water in the eastern part of this city for the last 15 years.It can be seen from these values that a_1 and a_2 vary little and that a_0 is diminishing every year.This shows that the cone of depression is expanding and that the boundary head H_0 at the radius of influence R is not a constant but it decreases every year,which is in agreement with the existing conditions.From the variable amplitude of these parameters and their laws, the variation between every two years is not obvious,and its tendency agrees with the hydrogeological conditions. This ensures that the application of this model for prediction is workable. It can be seen from Fig.1.1 and Tab.4.1 that the average error of the posterior prediction is rather small indeed and meets the requirement of precision for prediction.

Tab.4.1 Model parameters for the head variation of the Ⅳ aquifer of Labour Park

Time interval	Parameters			Mean Error checked every other year(m)	
	a_0	a_1	a_2	Absolute value	Arithmatic mean
From the second ten days in March to the second ten days in August (1969 | 1983)	−1.5139	−0.0492	0.7887	0.30	−0.29
	−2.5047	−0.0726	0.6626	0.38	−0.18
	−2.7994	−0.0685	0.6529	0.27	−0.16
	−2.9803	−0.0696	0.6494	0.38	−0.28
	−1.6897	−0.0824	0.8765	0.50	−0.14
	−3.7197	−0.0780	0.6271	0.39	−0.02
	−2.7017	−0.0410	0.7457	0.37	−0.20
	−3.0575	−0.0617	0.7214	0.39	−0.22
	−3.4606	−0.0770	0.6956	0.54	−0.49
	−3.8095	−0.0741	0.7093	0.64	−0.52
	−4.0802	−0.0991	0.6080	0.60	−0.56
	−5.2372	−0.1348	0.5220	0.57	−0.51
	−2.6747	−0.0521	0.8342	0.55	0.01
	−3.6903	−0.0512	0.7471	0.30	−0.07
	−3.1366	−0.0712	0.7490		
From the last ten days in August to the first ten days in March (1969 | 1983)	−1.8724	−0.0452	0.7571	0.27	−0.13
	−1.8335	−0.0429	0.7742	0.30	−0.06
	−1.7062	−0.0352	0.7837	0.32	−0.11
	−2.4160	−0.0481	0.7378	0.32	−0.24
	−3.1178	−0.0555	0.7019	0.43	−0.18
	−3.8279	−0.0592	0.6552	0.46	−0.15
	−3.2427	−0.0557	0.7074	0.36	0.12
	−2.1959	−0.0426	0.7816	0.46	−0.41
	−3.2125	−0.0505	0.7382	0.59	−0.55
	−5.8468	−0.0693	0.6067	0.44	−0.29
	−4.4804	−0.0658	0.6524	0.72	−0.71
	−6.3760	−0.0782	0.6019	0.38	0.35
	−5.5165	−0.0714	0.6318	0.35	−0.11
	−5.3279	−0.0701	0.6584		

5.Relation Between the Head and Deformations of Soil Layers

The deformation of the soil layer which causes the land subsidence is mainly due to the variation of the head.The aquifer in Shanghai area extends laterally much more than its thickness and therefore the theory of consolidation of one dimension is used to calculate the deformation of the soil layer.

The following equation is used to calculate the quantity of deformation of the clayey soil:

$$S_t=\frac{h}{1+e_0}\sum_{i=1}^{t}A_i\Delta P_i(1+K_0.\exp(C_iK_1(t-i)) \qquad (5.1)$$

The calculation of the quantity of deformation of the sandy soil is as follows:

$$S_t=\sum_{i=1}^{t}\frac{\gamma_w\Delta P_i}{E}h+S_0 \qquad (5.2)$$

where S_t is the accumulated deformation of the soil layer,in mm, up to the moment t;h is the thickness of the soil layer,in cm, half of which is taken when droinage occurs on both sides;e_0 is the initial pore ratio of the soil layer;ΔPi is the amplitude of the head variation during the time in terval between i-1 and i, in m;Ai is the compressibility coefficient of the soil layer; Ci is the consolidation coefficient of the layer;$Ko=8/\pi^2$; $K_1=30*86400\pi^2/h^2$;t is time,taking 10 days as a unit;γ_w is the density of water;E is the modulus of elasticity of the sandy soil,in Kg/cm^2;S_0 is the pre-accumulated deformation,in mm.

The parameters of Soil Mechanics in Eqs.(5.1),(5.2) can be found conversely from the long-term observed data of the head and the deformation of the layer which are measured in the field, and from the branch head-rising and branch head-falling periods, as we did for parameters in Eq.(2.1).Most of the correlation coefficients and these parameters obtained are over 0.95.

6.The Prediction of the Land Subsidence and Draw Up a Plan of Reasonable Development of Groundwater

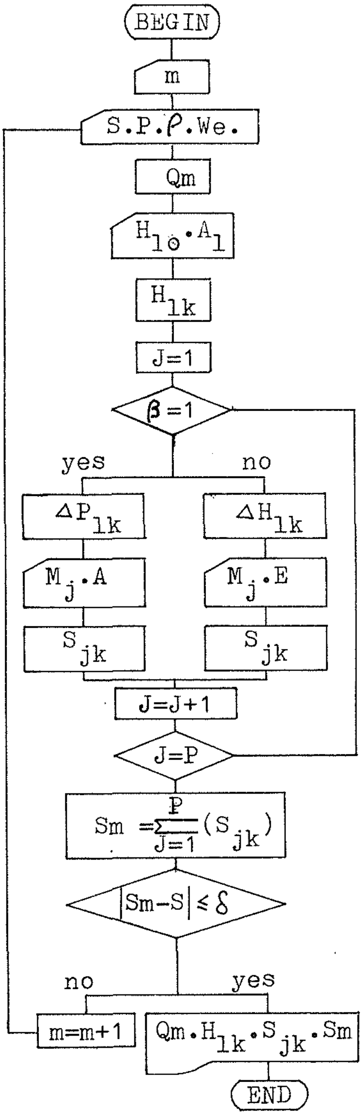
According to the development of industrial and agricultural production and long-term meteorological forecast, at the first the required amount of groundwater must be estimated for the prediction year,and using Eq.(2.1),the head variation can be predicted from the preliminary allocation of quantity of water which lie in different aquifers at different times.Again using Eqs.(5.1),(5.2),further the deformations of the soil stratums of the different layers can be calculated and after superposition the amount of land subsidence can be predicted.

If the predicted amount of land subsidence is bigger, the following measures must be taken:(1)The ratio of water supply in different aquifers must be adjusted.Equal quantities of groundwater developed in different layers of aquifer will have different effects on the deformation of the soil layer,due to the difference of hydrogeological conditions.(2)The amount of development must be reduced.This is very important,when considering it from the long-term point of view.Then the amount of land subsidence must be predicted according to the new allotment of water supply until a comparatively reasonable development plan of groundwater is obtained.The calculating procedure may be seen in Fig.6.1.

7.The Mathematical Model of Head-Variation Is Based on the Non-Steady Flow Motion Theory of Groundwater

The basic solution of groundwater non-steady flow motion equation—the source function of two dimensions is as follows:

$$H_t=\frac{1}{4\pi at}\exp(\frac{R^2}{-4at}) \qquad (7.1)$$



m: the type of water allocation

S: allowable deformation

P: the order of the aquifer

ρ: the order of the calculated layer

We: meteorology broadcasting data

Qm: plan for the allotment of the quantity of development

H_{lo}: the initial head of the l-th aquifer

A_l: the model parameters for the calculated head of the l-th aquifer

H_{lk}: the time sequence of the head of the l-th aquifer

β: identification marks for soils

ΔP_{lk},ΔH_{lk}:time sequence for the head amplitude of l-th aquifer

M_j: the thickness of the j-th layer

S_{jk}: time sequence for the deformation of the j-th layer

A: soil parameters of the clayey soil

E: soil parameters of the sandy soil

Sm: the accumulated deformation of the layer calculated from the m plan

δ: precision requirement

Fig.6.1 Block Diagram of the Calculating Flow Chart of the Plan

Under the condition that severed wells are under development, the head of the i-th observational well at the moment t through the time interval t is:

$$H_{i.t}=\frac{1}{\mu^*}\sum_{j=1}^{W}\int_0^t \frac{Q_{j.\tau}}{4\pi a(t-\tau)}\exp\left(-\frac{R_{ij}^2}{4a(t-\tau)}\right)d\tau \qquad (7.2)$$

The following equation is obtained by approximated simplification:

$$H_{i.t}=H_{i.to}+C_1X_{it}+C_2Y_{it} \qquad (7.3)$$

where $$C_1=\frac{1}{4\pi a\mu^*}, \qquad C_2=\frac{1}{(4\pi a)^2\mu^*} \qquad (7.4)$$

$$X_{it}=\sum_{j=1}^{W}\left(\sum_{m=0}^{t-to-1}\frac{Q_{j.t-m}-Q_{j.to-m}}{m+0.5}g_{jm}+\sum_{m=t-to}^{2(t-to)-1}\frac{Q_{j.t-m}}{m+o.5}g_{jm}\right)$$

$$Y_{it}=\sum_{j=1}^{W}R_{ij}^2\left(\sum_{m=o}^{t-to-1}\frac{Q_{j.t-m}-Q_{j.to-m}}{(m+o.5)^2}f_{jm}+\sum_{m=t-to}^{2(t-to)-1}\frac{Q_{j.t-m}}{(m+o.5)^2}f_{jm}\right)$$

in which $H_{i.t}$ is the head level of the i-th observational well at t,in m;$Q_{j.t}$ is the quantity of development of the j-th developping well (or injection well) during the time interval between t-1 and t,in 10,000 tons;R_{ij}^2 is the distance squared between the i-th observation well for the head and the j-th developping well,in m^2;a is the transmissibility coefficient of pressure; μ^* is the elastic release coefficient; W is the number of developping wells; to is the initial moment for calculation.

For g_{jm} and f_{jm}, let

$$Z_{jm}=\frac{R_{ij}^2}{4a(m+o.5)}$$

then

$$g_{jm}=\begin{cases}1\\o.7\\o.1\end{cases} \quad f_{jm}=\begin{cases}-1\\-o.28\\0\end{cases} \quad \begin{array}{l}0\leqslant \qquad <o.4\\0.4\leqslant Z_{jm}<2.5\\2.5\leqslant\end{array} \qquad (7.5)$$

With the model(7.3) and provided that the tranmissibility coefficient of pressure a and the elastic release coefficient μ^* are obtained by pumping test,the head variation at any point can be calculated according to the quantity of development of the individual developping well so that the long-term observation data are not needed.The result of the calculation is also acceptable.

8.The Reasonable Layout of Developping Wells

The model(7.3) describes the relation between the development points of groundwater and points of influence and therefore it can be used to analyse the reasonable distributions of developping wells and quantities of development so that a maximum allowable quantity of development of groundwater can be obtained under certain conditions of land subsidence.

Fig.8.1(a) and (b) are drawn according to the model(7.3). It can be seen from these two figures that the influence of the development points on the head in the surrounding area decreases rapidly with increase of distance and that the increase in quantity of water has no significant influence on the head. Within the radius of about 1 Km from the development point, the quantity of development has a comparatively big influence on the head variation, but within the radius of 1 to 2 km, this influence has greatly reduced, and beyond the radius of 2 km, its influence on the head variation almost vanishes. Therefore, when the position of the developping well is to be determined, it should be avoided that the distance between two wells is too close, so as to prevent a big fall of the head at a certain point between the two wells when equal quantities of groundwater are developed.

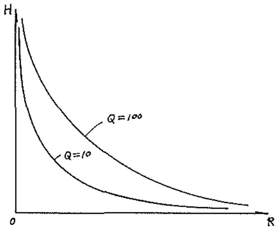


Fig. 8.1 (a)

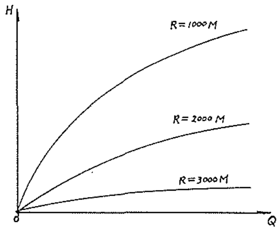


Fig. 8.1 (b)

The model(7.3) also creates a condition for us that we can consider the neasonable distributions of developping wells and quantities of development over an area.

According to the actual ground level of the city of Shanghai, a certain range of allowable land subsidence may be given and values of allowable drop of head at some controlling points can be determined accordingly. A reasonable distribution is that a maximum allowable quantity of development can be obtained when the head variations at the controlling points are not bigger than the allowable values. With this principle, a planning prohlom can be made:

To find that $Q(Q_1+Q_2+\dots+Q_W)$ reaches the maximum

Conditions to be satisfied:

$$\begin{cases} H_{i\cdot to}+C_1X_{it}+C_2Y_{it} \leqslant H_{i\cdot to}+\Delta h_i & (i=1,2,\dots,n) \\ Q_j \quad 0 & (j=1,2,\dots W) \end{cases} \tag{8.1}$$

where n is the number of controlling points of the head;

Δh_i is the value of allowable drop of head at the i-th controlling point.

Other symbols are the same as before.

This planning problom can be solved by the simplex method.

References:

1.Reports on the study of the land subsidence in the city of Shanghai (1962--1976).

2.Shanghai Geology, No.4,1981.

DEVELOPMENTS IN BOREHOLE EXTENSOMETRY

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Abstract

Progressive development of the deep-well extensometer over a period of 30 years facilitated the evolution of fundamental concepts and predictive capability in studies of aquifer-system compaction and land subsidence due to fluid withdrawal. Early taut-cable extensometers have been largely superceded by freestanding or counterweighted pipes, which typically have dimensionless strain resolutions of a few parts in 10^7, over depths of 200 to 1000 meters. Recording of high-resolution, low-drift compaction and expansion data is critically dependent on minimizing the effects of friction in all components of the system, and on virtually eliminating erroneous thermal and mechanical signals. Long-term records from standard extensometers, in conjunction with water-level piezometers, have made it possible to determine, in-situ, the aquifer-system properties that control land subsidence. Newer extra-high resolution extensometers permit definition of the compressibility and hydraulic conductivity of a thin, individual aquitard by means of a short-term pumping test.

Introduction

In three decades of studies by the United States Geological Survey (USGS) of land subsidence due to ground-water withdrawal, the progressive development of the deep-well extensometer has played a key role in the evolution of concepts, models, methodologies, quantitative results, and predictive capability The first part of this paper will briefly trace the development of the USGS instrumentation, the concepts suggested by the records obtained, and their ultimate verification through the development of predictive models capable of accurately matching the recorded subsidence history. The second part will define and elaborate what have come to be recognized as the fundamental requirements of a successful borehole-extensometer system.

Historical Development

The Survey's first borehole extensometer, or compaction recorder, was conceived by J. F. Poland, G. H. Davis, and J. H. Green, and was installed in the summer of 1955 in the San Joaquin Valley of California. Japanese investigators had used somewhat similar devices since the 1930's, but their achievements did not become known in the United States for about 30 years. Poland and others (1984) provide a very useful summary of the experience obtained with various types of extensometers developed in Japan and Mexico, as well as in the United States. To the extent permitted by considerations of internal completeness and continuity, the present paper avoids repetition of the material presented by Poland and others (1984).

Early extensometers--The first USGS extensometers used a tensioned cable to measure changes in the distance between a subsiding land surface and a bottom-hole anchor set near the base of the pumped aquifer system. The extensometer cable ran over two fixed pulleys to the tensioning counterweight, and its apparent movement relative to the instrument platform, or datum, was recorded by a standard water-stage recorder (Lofgren, 1961). Within a few years, these simple devices proved conclusively, for the first time, that the land-surface subsidence measured by repeated leveling surveys was attributable to compaction of the confined aquifer system; further-

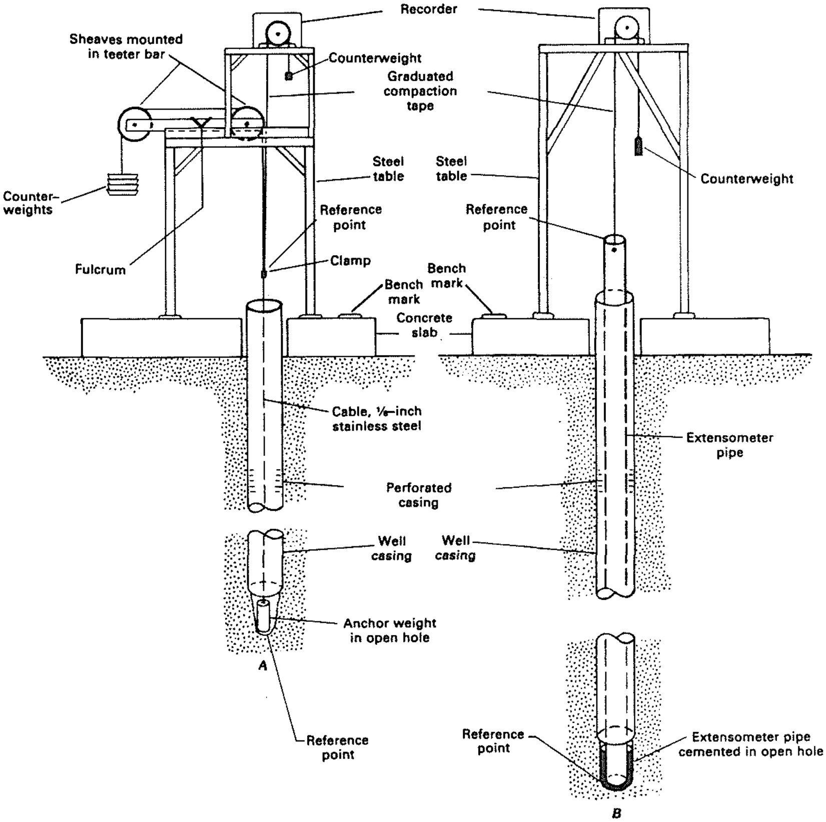

Figure 1

more, they demonstrated that the time distribution of such compaction was was related in a general, though not always simple, way to be the time time distribution of head decline in the pumped aquifers.

Data, concepts, and models--The close correlation, over a period of about 3 years, between head change in the pumped aquifers and aquifer-system deformation measured by a 460-m cable extensometer is shown in figure 2A. The observed head change is a direct measure of the change in effective stress in the aquifers and of change in stress applied to the interbedded aquitards. Therefore, these data may be plotted one against the other in the form of a stress-strain diagram (fig. 2B). The reciprocal slope of the stress-compaction trend line is S_k, the component of the aquifer-system storage coefficient attributable to deformation of the matrix, or granular skeleton (Riley, 1968). For this site $S_k = 3.0 \times 10^{-3}$. The hysteresis shown by the somewhat open loops reflects the phase lag between applied stress change and aquifer-system deformation. This phase lag results from the impedence to flow of water in and out of storage in the aquitards, as a function of their low permeability and appreciable thickness. The deformations seen here are fully elastic and recoverable, because the stresses do not exceed the maximum past stress (preconsolidation stress). The average vertical compressibility for the 335-m thickness of interbedded aquifers and aquitards at this site

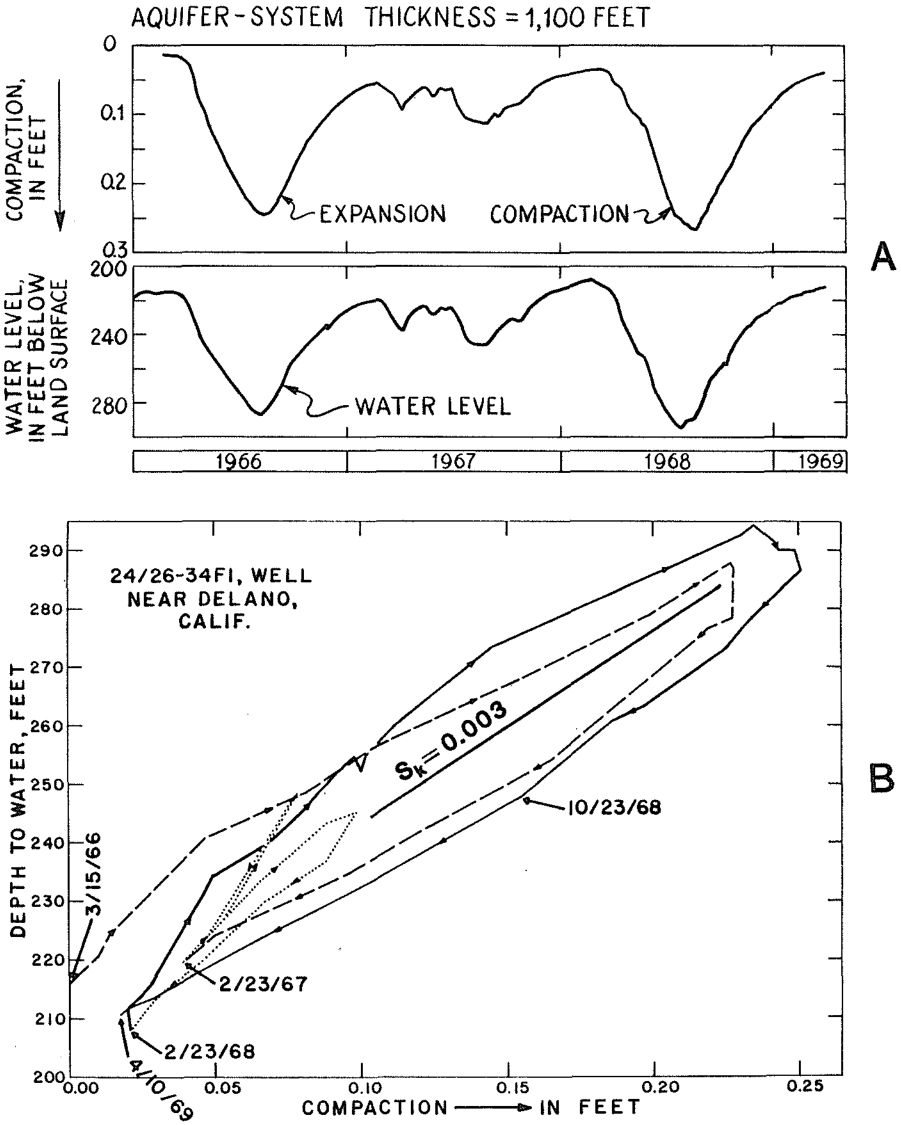

FIG. 2 Head change and recorded aquifer-system deformation: A. Fluctuations in head (stress) and thickness of the confined aquifer system; B. Drawdown-compaction (stress-strain) relationship.

is 9.1 x $10^{-10} m^2/N$, or 8.9 x 10^{-6} m^{-1} when expressed in terms of S_{ske}, the skeletal component of elastic specific storage (Riley, 1969).

Figure 3 illustrates a different situation in which each year the maximum past stresses in at least some portions of some aquitards are exceeded for part of each pumping season. This results in large increments of nonrecoverable deformation during the deeper part of each annual drawdown cycle--those parts below the line B-B' on the stress-

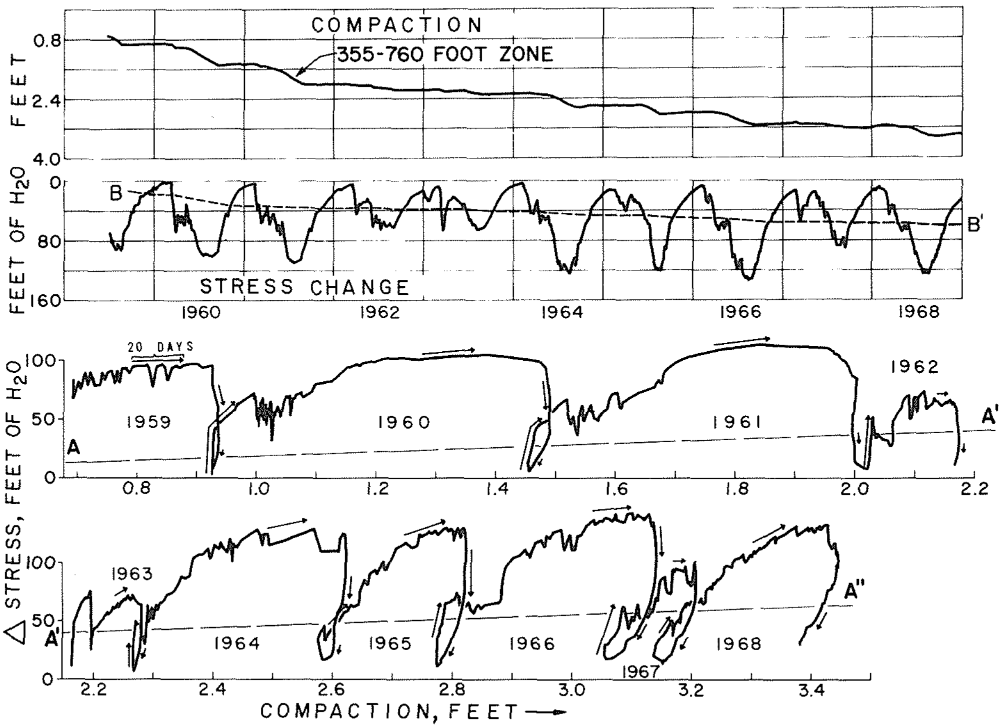

FIG. 3 History of compaction and stress change, and the relationship between stress change and compaction near Pixley, California.

change hydrograph. If this process continues long enough or demands for water are reduced, the response eventually becomes entirely elastic, as in figure 2. On the stress-compaction graph in figure 3 the line A-A'-A" defines the upper limit of stress fluctuation within which the deformation is essentially elastic.

Using the several slopes available from stress-strain plots of this kind, Riley (1969) showed that it is possible to estimate not only the elastic storage coefficient, but also the much larger inelastic or virgin storage coefficient, and the critical threshold value or preconsolidation stress at which the response changes from elastic to inelastic. If additional geologic data on the number and thickness of aquitards are available, the specific storage, or compressibility, can be estimated; in addition, the time-response characteristics of the system can be used to derive an average vertical permeability for the aquitards (Riley, 1969).

Helm (1975, 1976, 1977, 1978) incorporated this conceptual model and the in-situ parameters derived therefrom into a numerical aquitard-compaction model which reproduced very closely the known history of subsidence at a number of sites in California. His method facilitates an iterative optimization of the parameter values, which initially may be quite crude. Helm's one-dimensional model is a straight-forward application of Terzaghi's theory of time-dependent soil consolidation, and provided the first clear-cut demonstration that all observed details of the aquitard compaction process could be accurately simulated using this theory. The parameter inputs required by the model

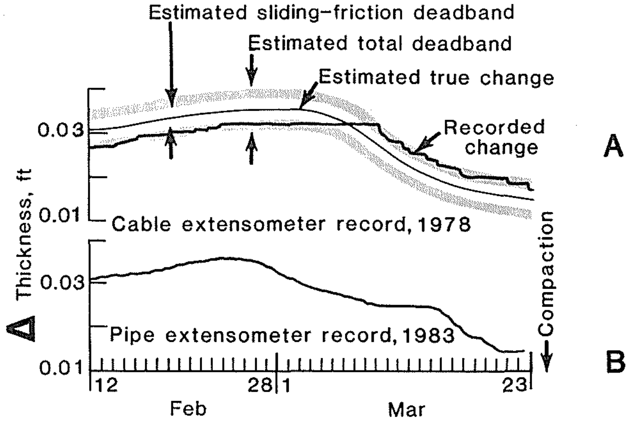

FIG. 4 Typical field records of rebound and compaction of an extensometer installation, 260 m deep, near Eloy, Arizona: A. Record produced by cable element; B. Record produced by counterbalanced pipe element.

include the number and thickness of the aquitards in the compacting aquifer system as well as preliminary estimates of their average permeability and specific storage (both elastic and inelastic). The model is driven by a more-or-less detailed history of the changes in applied stress, as defined by records (or projections) of the head changes in wells tapping the permeable strata in the aquifer system. Outputs include a detailed compaction history (or projection), refined average values of vertical hydraulic conductivity and of inelastic and elastic specific storage, and the preconsolidation stress and distribution of residual excess pore pressure at any time. Without high quality extensometer records, this model could not have been verified, and recognition of the concepts on which it is based might have been substantially delayed. For a summary of 25 years of extensometer data from central California, and analysis of the application of Helm's model to these data see Ireland and others (1984).

Errors due to down-hole friction--A typical record from a cable-extensometer recorder chart is reproduced in figure 4A. In 1978 this extensometer installation, 260 m deep with a 20.9 cm casing, produced a stepped record characteristic of moderate cable-casing friction. The average amplitude of the near-vertical steps (about 0.6 mm) indicates the approximate width of the stick-slip portion of the total frictional deadband. The upper and lower stick-slip deadbands are shaded in figure 4A. A reversal in the trend of deformation displaces the recorded change across the deadband, with the result that the maximum (or minimum) is truncated and an apparent phase lag is introduced in the record.

It should be noted that the magnitude of frictional steps recorded in normal operation cannot be used as a reliable indicator of the deadband width. These steps reflect the difference between static and sliding friction. Thus, the rest point at the end of a stepwise adjustment of extensometer length constitutes a point approximately on the outer edge of the sliding-friction deadband, which may represent more than half the total (sliding plus static) deadband width (fig. 4A). A good extensometer operating with no discernible static (stick-slip) friction may still have a significant sliding friction deadband.

Errors resulting from the frictional deadband may significantly limit and distort the information content of the record, especially where the forcing function (aquifer head change) contains frequent episodes of drawdown and recovery. The deformation responses to such reversals in trend are particularly useful for defining the hydrodynamic lag in the dissipation of excess pore pressures. In the application of Helm's aquitard-compaction model to stress-strain data it is principally the response to trend reversals that constrains the values of hydraulic diffusivity and preconsolidation stress.

In order to permit a clear distinction between attenuation and phase shift due to hydrodynamic lag, versus similar effects due to instrument friction, the instrument characteristics should be defined through a deadband test. Such a test is readily accomplished by alternately increasing and decreasing the uplift force on the extensometer cable sufficiently to move the cable past major points of friction against the casing. Typically, the process requires producing a strain of 1-to-10 x 10^{-5} in the cable. As the disturbing force is gradually removed, the cable springs back toward its "true" length, but does not fully attain it. Thus the recorder pen comes to rest above or below the "true" position, depending on the direction from which the "true" position is approached. The rest points constitute the approximate upper and lower limits of the sliding-friction deadband. The midpoint between them may be taken as representative of the undisturbed length of the cable under the applied counterweight tension. If the initial response to an expected reversal in stress--for example, the beginning (or end) of a pumping period--is to be accurately recorded, the extensometer may be manually preloaded to the high (or low) side of the sliding-friction deadband, as in performing half of a deadband test. It is then ready to respond to the first small increment of compaction (or expansion).

Pipe extensometers--In the late 1960's, Poland's group began experimenting with free-standing pipe extensometers (fig. 1B), having learned of the Japanese success with this type of design. Initial experiences indicated that relatively large-diameter extensometer pipes (2 or 2 1/2-inch nominal diameter) could generate virtually step-free records operating without centralizers to depths of several hundred meters in relatively small (4-inch, nominal) casings (Poland and others, 1984). This tends to be true despite the fact that in a well more than perhaps 70 m deep the pipe will bend enough under its own weight to induce some frictional contact with the casing. It is surmised that the heavy-duty couplings in the pipe string function as relatively low-friction centralizers in small-diameter casing. In the Houston, Texas, area a free-standing pipe 936 m deep is reported to be generating a very good record (Robert Gabrysch, oral commun., 1984). In this installation the element is 2-inch (60.3 mm outside diameter) steel tubing in a 5 1/2-inch (118.6 mm) inside diameter casing. The radial clearance between the couplings and the casing is 0.898 inch (22.8 mm).

Smaller diameter pipes in larger casings are less successful, because they bend enough under their own weight to exert large friction-inducing lateral forces against the lower parts of the casings. The friction problem is exacerbated if the well is significantly out of plumb, causing a long length of pipe to bear against the low side of the casing. Poland and others (1984) reported that a 1 1/2-inch (48.3 mm outside diameter) pipe in a 10 5/8-inch (260 mm inside diameter) casing 381 m deep produced a stepped record with vertical offsets of 0.15 to 0.3 mm. The actual width of the total frictional deadband probably was at least 1.0 mm.

In most situations, especially with extensometers of more than 200-m depths, the pipe tends to perform better than the cable because its much

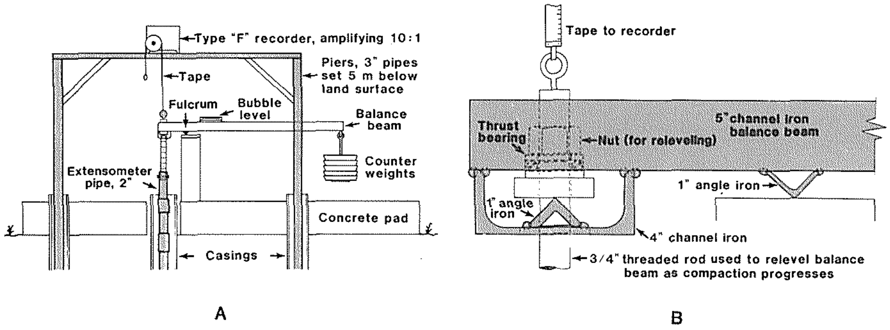

FIG. 5 Counterbalanced pipe extensometer: A. Schematic of components at land surface; B. Detail of knife edges and bearing plates supporting balance beam and extensometer pipe.

greater cross-sectional area enables it to overcome downhole friction with minimal induced change in length. It should be noted, however, that in a well whose diameter and straightness are sufficient to eliminate casing-cable contact throughout its depth, a taut cable or even a light wire or tape will provide an excellent extensometer element. Riley (1970) used a 50-meter invar surveying tape as the extensometer element in a virtually friction-free installation that had a resolution of ± 1μm.

High-resolution extensometers--In an effort to improve the record illustrated in figure 4A, the cable at this site was replaced with a 2-inch pipe, and an asymmetric counterweighted lever (balance beam) was used to support the upper end of the pipe. The installation was based on the design schematically illustrated in figure 5. The 90 kg counterweight, acting on the 8-to-1 mechanical advantage of the balance beam, suspended the upper half of the extensometer pipe in tension and minimized frictional contact with the well casing. The resulting improvement is illustrated by the step-free record shown in figure 4B for February and March, 1983. The total deadband is estimated to be about 0.06 mm.

High-resolution records obtained with pipe extensometers encouraged the author to undertake to record the minute deformation of a thin overconsolidated confining clay, in response to nearby pumping from an underlying aquifer. The total compaction of the 4.3-m thick clay bed was not expected to exceed 0.5 mm, which suggested a desired resolution of 0.005 mm and a thermal drift of less than 0.025 mm per month. These criteria translate into a frictional force between pipe and well casing of no more than 30 newtons (N), and an uncompensated change in average temperature along the 32-m pipe of no more than 0.07°C.

The design adopted was a free-standing pipe extensometer incorporating special features for minimizing down-hole friction and the effects of near-surface soil instability and temperature change (fig. 6). A frictionless electronic linear-motion transducer (LVDT) sensed land-surface movement, which was recorded on magnetic tape by a digital data logger (fig. 6A). To prevent changes in temperature and bouyant support of the pipe due to drawdown in the casing when the aquifer is pumped, a cement plug capped by an impervious but flexible polymer grout was placed in the bottom of the well (fig. 6B). A heavy vinyl sleeve, pressurized and internally lubricated by glycerine, preserved the integrity of the grout seal around the lower part of the 60.3-mm extensometer pipe without transferring frictional loads from the 102-mm plastic casing to the pipe

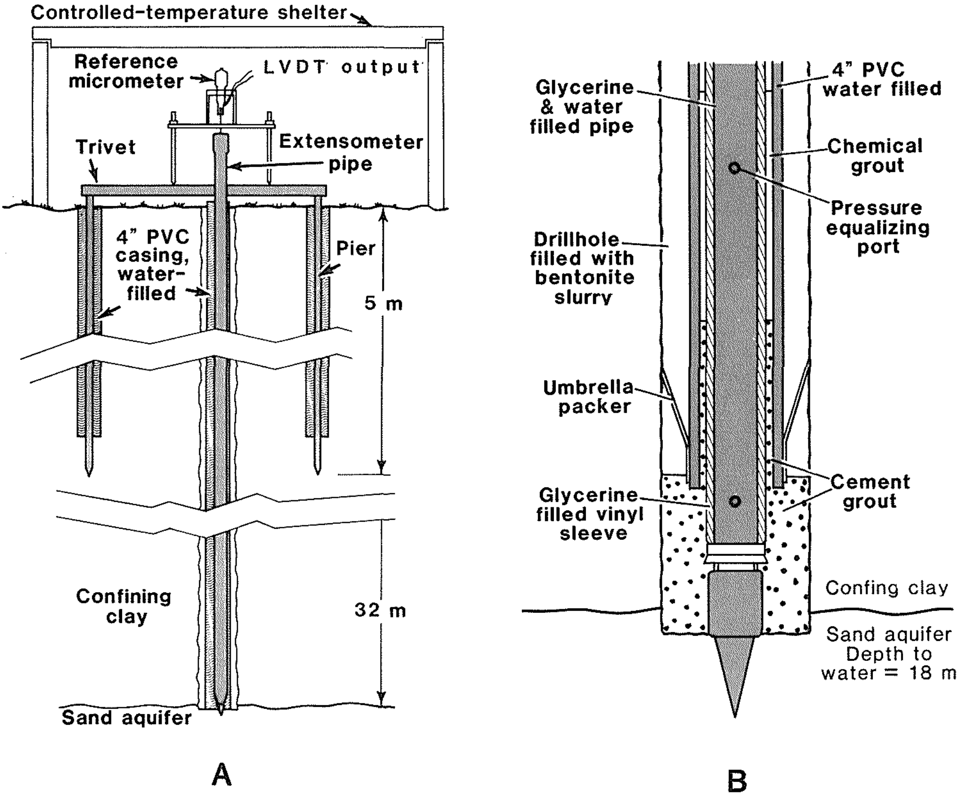

FIG. 6 Extra-high resolution extensometer used in pumping test near Lake City, Florida: A. Surface and subsurface features; B. Detail of extensometer base.

(fig. 6B). Frictional coupling from borehole to casing and casing to pipe was minimized by using PVC plastic casing, which has a smooth, slippery surface.

To promote a constant-temperature environment, the extensometer pipe, instrument piers, well casing, and pier casings were filled to land surface with water. Above ground components of the system were covered by an insulated heated shelter, thermostated to ± 0.2°C.

The elastic compaction (0.55 mm) and rebound of the confining bed measured during 14 days of pumping at $2.0 \times 10^{-4} m^3 s^{-1}$ and 15 days of recovery are illustrated in figure 7. The raw data from this extensometer and two similar units extending to depths of 36.6 m and 41.5 m show no discernible deadband effects at a resolution of 0.002 mm. Responses of about 0.01 mm to diurnal barometric fluctuations are clearly defined in the raw data, but do not show at the scale of figure 7. However, temporary compaction due to rainfall loading is readily apparent (fig. 7A).

Application of Helm's aquitard-compaction model to the data from this pumping test produced a plot of computed compaction and rebound (Helm, 1977, personal comm.) that is shown as the dashed lines on figures 7A and 7B. From this simulation the following aquitard properties were derived: vertical hydraulic conductivity is 2.7×10^{-5} m/day; matrix compressibility (expressed as the skeletal component of specific storage) is

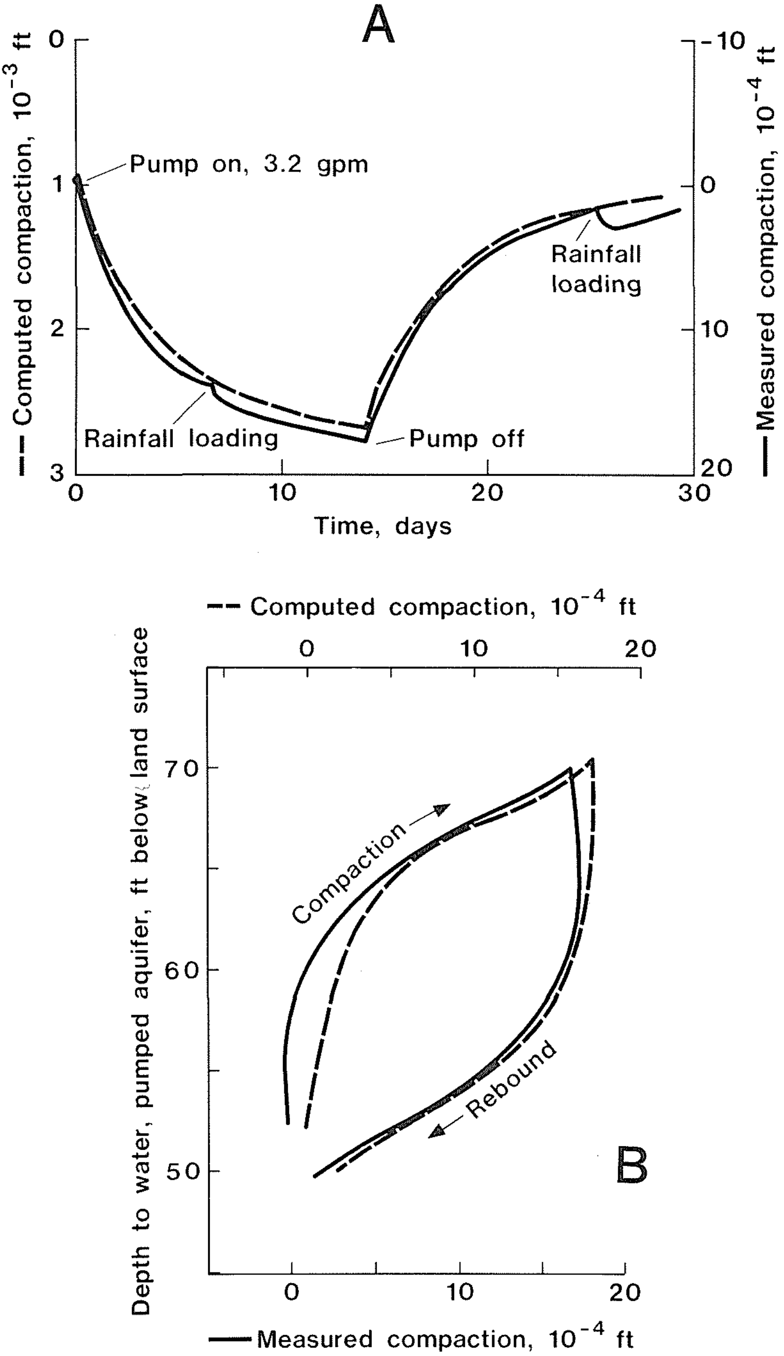

FIG. 7 Elastic compaction and rebound of the 4.3-m confining clay during a pumping test near Lake City, Florida: A. Measured and computed history of vertical deformation; B. Measured and computed stress-strain relationships.

$3.9 \times 10^{-5}\ m^{-1}$; the aquitard is overconsolidated to a stress level that exceeds the prevailing overburden stress by at least 6 m of water head.

Comparative stress-strain plots depicting the measured and computed transient response to pumping drawdown and recovery (fig. 7B) are sensitive indicators of the accuracy of this simulation process; their usefulness derives from the fact that the stress-strain relationship is strongly influenced by hydraulic conductivity during the early stages of rapid drawdown and recovery, but is controlled largely by specific storage during later stages, when head change and compaction are proceeding very slowly.

Stress-strain plots based on the measured and computed deformation shown in figure 7A were, for most part, so nearly identical that the plot of computed response was offset to the right by 2×10^{-4} ft (0.061 mm) in drafting figure 7B, to facilitate distinction between the two curves. However, the computed curve departs significantly from the measured curve during the early phases of drawdown. This departure is attributable to the immediate, mechanically-coupled aquitard response to flow-induced centripetal strain in the pumped aquifer. Wolff (1970) observed such strains, as manifested at land surface, and also measured the associated transitory increase in pore pressure ("reverse water-level fluctuation") in the aquitard. The small (6×10^{-3} mm) initial expansion of the aquitard seen in figure 7B is believed to be the first direct measurement of the transverse (vertical) extension associated with radial compression near a pumping well. The one-dimensional aquitard-compaction model simulates neither the horizontal compression due to radial flow nor the accompanying vertical extension in accordance with Poisson's ratio; therefore, the computed deformation does not track the measured deformation during the first minutes of rapidly changing horizontal strain following starting (or stopping) of the pump.

The absence of a progessive departure between the computed and measured stress-strain curves (fig. 7B) implies that the linear stress-strain law incorporated in the simulation is a reasonably accurate representation of the actual deformation process, within the range of stress applied. It also implies a virtual absence of thermal or other forms of instrument drift over the 29-day duration of the test. Thus the experiment demonstrated the feasibility of constructing a nearly frictionless and drift-free extensometer sensitive enough to permit determining, in situ, the hydraulic conductivity and compressibility of a single stratum, by means of inducing a relatively small, controlled stress for a short period of time.

Design and Operational Considerations

The design and operational requirements for an ideal borehole extensometer system may be addressed in terms of the six principal components of the system:

1. The base, or anchor, which constitutes the bottom-hole reference point.
2. The casing of the extensometer well.
3. The instrument platform, which constitutes the extensometer datum at land surface.
4. The extensometer element (pipe or cable).
5. The above-ground mechanisms that support the element.
6. The measuring devices that record movement of the land-surface datum.

These six components are discussed in some detail in the following paragraphs, with the goal of providing a summary of fundamental design criteria and operational procedures, many of which have not previously been explicitly described.

Extensometer base--In the most general case, in which total compaction is to be recorded, the base or anchor of the extensometer (fig. 1) should be established somewhat deeper than the maximum anticipated depth of induced pore-pressure decline, which may be substantially greater than the maximum depth of producing wells. If information on the vertical distribution of aquifer-system compaction is desired, additional extensometers may be established with bases set at the boundaries of subintervals of interest within the aquifer system. Stability of the anchor requires that it not settle into soft or disturbed sediments at the bottom of the borehole, and that the bottom of the hole not heave up because of readjustment of stresses after drilling. Experience suggests that these problems, if encountered at all, tend to be self-limiting, of brief duration, and readily identifiable on the continuous recorder charts. As a conservative approach the anchor should be set on or in a cement plug 3 to 5 m thick.

A potentially more insidious problem is the transmission of gradually increasing downward stress from within and above the interval of compaction downward to the environment of the anchor, through skin friction with the casing string (the pile effect). The process is comparable to the development of what soils engineers term "negative skin friction" on a pile. These downward directed skin-friction forces acting on a pile commonly develop if a pile is driven through a soft soil layer that is compacting because of reduction in pore pressure or the placement of a fill. According to Lambe and Whitman (1969) a relative displacement of about 2.5 cm is sufficient to mobilize fully the skin friction of soil on a pile. Johannessen and Bjerrum (1965) describe a field test in which 55 m steel piles were driven through a soft marine clay, which was then loaded with 10 m of fill. The surface of the fill subsided about 1.2 m, reflecting consolidation of the clay. Negative skin friction produced an overall shortening of the pile of 14.3 mm and forced its tip into the underlying bedrock. Stresses in the pile near the tip were calculated to be of the order of 2 x 10^5kPa (approaching the yield strength of steel), and total negative skin friction was about 2.2 x 10^6N.

The severely wrinkled, crushed, and sheared well casings commonly encountered in subsiding areas provide abundant evidence that these casings can transmit substantial downward forces to the sediments adjacent to and immediately below the bottom of the casings. The magnitude of the transmitted force is limited by the ultimate strength of the casing, which typically may be in the range of 1-to-3 x 10^6N. If an existing well is to be converted to an extensometer installation, it should be deepened about 15 m to place the cement plug and subsurface benchmark below the bulb of sediments that are stressed by the deforming casing. A newly constructed extensometer well should incorporate at least one slip joint in the casing string about 10 m above the bottom, to minimize downward transmission of stress. Other benefits of slip-joint construction are discussed below.

Well casing--Negative skin friction effect on the load-bearing casing tends to redistribute stresses adjacent to the borehole so as to reduce somewhat the compaction within the interval of pore-pressure decline, induce compaction below that interval, and induce expansion above that interval. The net result is that the extensometer progressively under-records aquifer-system compaction and land-surface subsidence. The problem can be minimized or eliminated by constructing an extensometer well using telescoping slip joints installed in the casing string near the upper and lower boundaries of the compacting interval, and within the interval at spacings suggested by the stratigraphy and the magnitude of anticipated compaction.

Poland and others (1984) illustrate various telescoping casing and slip-joint constructions used in Japan, Mexico, and the United States. In addition to minimizing the role of the casing as an unwanted load-bearing member in the system being measured, these constructions enhance the longevity of the installation by postponing the onset of casing deformation and failure.

A newly constructed extensometer well can be completed using a heavy bentonite mud especially prepared for optimal sealing and plugging. An umbrella or basket packer near the bottom of the casing (fig. 6B) is necessary to hold the mud in the annulus when the casing is cleaned out. Such a completion procedure may be expected to delay and reduce the effects of skin friction, by reducing the tendency for the formation to cave and squeeze around the casing, and by providing a lubricated contact between the casing and formation. Where an annular seal is required to prevent vertical migration of poor quality water, using these specialized muds in preference to cement grout should be considered. If cement grout must be used, it should be emplaced opposite formation intervals that are expected to experience minimal compaction, and it should be of low density so as not to settle through the mud column.

Extensometer datum--Using a concrete pad to support the instrument platform, as was done with early installations (fig. 1), allows moisture, temperature, and biological changes in the soil to disturb the datum. Such changes usually are concentrated in the upper 4 to 6 m of soil. Their effects can be minimized by establishing the instrument platform on pipe piers, typically 2 to 4 inches in nominal diameter, that are forced into the bottom of oversized holes bored 5 to 8 m below the surface (figs. 5A and 6A). The pier wells are cased to ensure permanent decoupling of the piers from the shallow soil. Separation of the piers from the extensometer well by a radial distance of 1.0 to 1.5 m places the datum outside the cylinder of local disturbance that may develop if the well casing exhibits increasing protrusion during the life of the installation.

Seasonal and diurnal temperature variations in the pier wells are comparable to those in the extensometer well, and induce length changes in the piers that are comparable to (and therefore tend to cancel) the temperature-induced length changes in the corresponding upper 5 to 8 m of the extensometer element. Thus the piers transfer to the instrument platform above land surface a stable, approximately temperature-compensated, near-surface datum that may be expected to remain nearly invariant relative to the top of the compacting interval. For critical, ultra-sensitive measurements it may be necessary to control temperatures within the instrument shelter to within ± 0.5°C, or better.

A concrete pad, decoupled from the extensometer and pier wells, may be employed as the foundation for the fulcrum that supports the counterweight lever or balance beam. In a properly designed system, vertical movement of the fulcrum due to pad instability will cause a balance beam to tip but will not impart any spatial disturbance or change in load to either the extensometer element or the instrument platform.

Extensometer element--The most difficult problem typically encountered in obtaining optimal performance from a deep-well extensometer is ensuring that the measuring cable or pipe maintains an invariant length. Virtually all wells deeper than about 50 m depart from straightness sufficiently to force contact between the measuring element and the well casing. During compaction, downhole friction between the element and the shortening well casing induces changes in the stress distribution in the element. These stress changes characteristically produce time-varying and more-or-less indeterminant length changes that degrade and distort the record of aquifer-system deformation. Downhole friction typically is the limiting factor in determining extensometer resolution. In extra-sensitive installations, temperature and bouyancy effects may also become significant.

Abandoned production wells that have often been used for low-cost cable extensometer installations were seldom straight when first drilled and subsequently were subject to severe casing deformation and failure under the stresses of aquifer-system compaction. Under these conditions, the extensometer cable typically is seized by the casing at friction points within and above the compacting interval, and is temporarily constrained to move downward in partial or total compliance with the deforming casing and subsiding overburden; the result is that little or no movement of the cable relative to the instrument platform is recorded for a time. Continuing compaction and shortening of the casing below a locked friction point results in concomitant shortening of the adjacent section of cable and, therefore, a reduction in the tensile stress in that part of the cable. The accumulating imbalance between the progressively diminishing stress in the lower part of the cable, below the friction point, and the constant counterweight stress in the upper part of the cable eventually overcomes the friction in a stick-slip dislocation recorded as an abrupt stair-step in the compaction record (fig. 4A). In a typical cable installation the intermittent adjustments across many such friction points combine to generate a record consisting of numerous steep or vertical steps of somewhat irregular amplitude and spacing. A chain reaction is often discernible, as the slippage across one friction point disturbs the stress distribution across adjacent points and causes the readjustment to propagate up and down the cable. Step amplitudes of 2-to-5 X 10^{-6} times the cable length must be considered typical for cable extensometers; however, frictional stepping can be an order of magnitude more severe in wells more than 500 m deep that depart from verticality by 5 degrees or more.

For a given amount of aquifer-system compaction, the unbalanced force available to overcome friction is the product of the shortening of the locked portion of the extensometer element, its elastic modulus, and its cross- sectional area, divided by the length of locked element that is subject to stress reduction due to shortening. Thus, small cable diameter and a deep extensometer contribute to aggravated frictional stepping. Contrary to possible intuition, the use of heavier counterweights in a cable system does not increase the force available to overcome friction, but does pull the cable more tightly against bends, breaks, and rough surfaces in the crooked casing, thus tending to increase downhole friction. In general, counterweight mass should not greatly exceed that required to support the weight of the cable in a stable, taut alinement.

The standard cable used in USGS extensometers is a nominal 1/8-inch (3.175 mm) diameter stainless steel, 1 X 19 strand, reverse-lay aircraft control cable, having a cross-sectional area of 6 mm^2 and a weight of 0.047 kg/m. This cable resists corrosion, does not untwist under tension, and has shown no signs of fatigue elongation (Lofgren, 1969). Counterweight loads of 6 to 8 kg per 100 m of cable probably are most appropriate, although loads several times greater than that have often been used.

The use of a pipe as the extensometer element, although substantially more expensive than a cable, offers a major advantage in overcoming downhole friction, because of the pipe's much greater stiffness. A typical pipe element of nominal 2 1/2-inch diameter (730 mm outside diameter) has a cross-sectional area about 180 times larger than the standard 1/8-inch cable. Some of this intrinsic advantage is lost, however, because in deep wells the pipe bends enough under its own weight to assume a sinusoidal or spiral configuration that forces it into frictional contact with even a straight, vertical casing. Relatively small pipes operating in relatively large casings bend enough to exert large friction-producing lateral forces against the lower parts of the casings. Better results are achieved when the radial clearance between pipe couplings and casing is only 10 to 25 mm. If a well deviates appreciably from the vertical, the weight of the extensometer pipe will tend to force long sections of the pipe against the lower side of the bends in the casing, creating substantial friction.

In many cases the performance of the pipe extensometer can be greatly enhanced by suspending some or most of the weight of the pipe from a counterweighted lever system (fig. 5). If the hole is approximately straight, and especially so in its upper portion, most of the weight may be counterbalanced, so that most of the pipe hangs in tension and does not tend to bow against the casing. If there are one or more substantial bends in the lower portion of the casing, the point of neutral pipe stress may be positioned near or somewhat below the uppermost bend so as to minimize the weight forcing the pipe against the outside of the bends. The most intractable situation arises when there is significant bending in the upper as well as lower parts of the casing. A compromise must then be attempted between excessive compressional flexure in the deep part of the pipe versus excessive tensional pull against intervals of maximum curvature in the upper part of the well. From these considerations it is evident that a directional, or deviation, survey of the well bore can be very helpful in determining the pipe specifications and approximate uplift force. In some cases it may be desirable to employ a tapered element composed of several segments of different sized pipe, the diameter increasing with depth (Poland, and others, 1984).

Changes in extensometer length due to the typically small temperature changes below the compensated near-surface interval are, as a general rule, important only in special-purpose, extra-high precision installations. However, if a perforated well casing allows air flow in and out of unsaturated sand or gravel layers at depth, the resulting "inhaling" of surface air during periods of relatively high barometric pressure can produce substantial changes in the temperature and length of the extensometer. In other situations, large pumping drawdowns of initially shallow water levels may expose portions of the extensometer to temperature changes large enough to induce significant length changes. The effects of air or water movement in the well casing can be controlled by sealing the well to prevent fluid communication with the surrounding sediments. In some cases, collection of temperature data at various depths in the borehole and pier wells may be essential to correct for uncompensated thermally induced length changes. Use of electronic data loggers as described in a later section, greatly facilitates the collection of temperature data.

Length changes due to change in buoyant support can also be controlled by sealing the well, if desired. It has been assumed that buoyancy-induced changes, if significant, could be corrected computationally through application of Archimedes' Principal. Such corrections have not been routinely made. In most installations calculated buoyancy effects can be reduced to tolerable magnitude (<0.5 mm) by ensuring fluid communication between the water in the pipe and that in the casing, so that the levels fluctuate together. To the best of this writer's knowledge, no controlled field experiments on buoyancy effects have been performed.

For reasons of economy it has been common practice to screen or perforate extensometer wells in one or more depth intervals deemed to be representative of the average head within the compacting aquifer system; thus both the head-change and deformation data necessary for defining stress-strain relationships could be obtained in the same well. This practice not only produces some bouyancy errors, but also may cause significant thermally-induced changes. Temporal variations in vertical head distribution may alter the velocity of interaquifer flow within the well bore enough to cause substantial changes in the thermal environment of the extensometer. The preferred practice of minimizing water-level fluctuations in the extensometer well and installing a nearby array of point-sensing piezometers eliminates these sources of error and permits a more rigorous definition of the vertical distribution of stresses causing deformation.

Support mechanisms--If a counterweighted balance beam is employed to maintain tension on a cable or to support part of the weight of a pipe, the active pivots should be knife edges supported by flat horizontal steel plates (fig. 5B). The use of flat supporting surfaces allows for flexibility in positioning the knife edges, to accommodate individual variations in installations. To achieve the required neutrally stable balance configuration, all pivots must be located in a commmon plane. If fixed rather than freely suspended counterweights are used, their center of gravity should fall in the plane defined by the center pivot and the knife edge supporting the pipe or cable. These precautions ensure that the tipping of the balance beam that occurs with subsidence of the fulcrum does not alter the system geometry in such a way as to change the uplift force on the extensometer element. Vertical movements of the fulcrum caused by instability of the concrete pad will make the balance beam tip, but will not affect the compaction record. As subsidence accumulates, the balance beam is releveled when tipping exceeds about 5 degrees, by adjusting the suspension nut (fig. 5B) or rotating the cable-suspension pulleys (fig. 1A) during regular servicing of the instrument.

When a substantial fraction of the weight of an extensometer pipe is to be suspended from a balance beam, it is convenient to use an asymmetric lever that provides a mechanical advantage in the range of 8:1 to 12:1 (fig. 5A). The required uplift force (typically 1000 to 5000 kg) can then be developed with a manageable counterweight mass, and loads on the fulcrum and pad are minimized. No attempt should be made to use the long moment arm as a means of amplifying the compaction signal for recording purposes, because to do so would contaminate the compaction record with an amplified recording of pad instability.

The use of a counter-balance system incorporating a mechanical advantage of about 10:1 confers a very important operational advantage, by facilitating the controlled application and removal of large and precisely variable uplift forces on the pipe. This makes it easy to test the ability of the extensometer to return accurately or approximately to an original length after substantial induced excursions in both directions. Such deadband tests are essential to quantifying the frictional properties of an extensometer, to characterizing its overall performance, and to developing confidence in the reliability of its record. Optimal performance commonly results from a trial-and-error process of adjusting the counter-balance force in increments and testing the frictional deadband with each different counterweight load.

As the land surface subsides around the extensometer element, progressively increased protrusion of the pipe or cable must be accommodated. With a cable system the counterweight suspension need only be shortened occasionally, as required. Protrusion of a pipe is initially accommodated by screwing down the releveling nut on the threaded suspension rod (fig. 6). With continuing compaction, however, short sections of the pipe must eventually be cut off, or preferably unscrewed, from the top of the pipe. Continuity of a high-precision record is best assured by finishing the upper end of the pipe with a prefabricated assembly consisting of several short sections of pipe (about 20 cm long) connected by threaded couplings (fig. 5A). Each section contains a precisely located hole for the suspension pin that ties the pipe to the threaded suspension rod and thence to the counter-balance lever. If a free-standing rather than counter-balanced pipe is used, the pin serves as the anchor for the tape that drives the compaction recorder (fig. 1B). The holes are drilled in a machine shop at accurately determined and recorded spacings, and a vertical line is scribed on the assembly to insure that the pipe section and couplings are not rotated out of their original alignment. The assembly is fitted to the upper end of the extensometer pipe and positioned so that the holes and suspension pin are parallel to the counter-balance lever. This arrangement provides two degrees of freedom in coupling the pipe to the lever and thus accommodates minor misalignment. As subsidence progresses individual sections are removed from the top of the assembly as necessary, and the threaded suspension rod is pinned to the pipe through the next lower hole. The predetermined hole spacing then becomes a constant added to the record of accumulated compaction.

Measuring devices--Continuous analog records of aquifer-system compaction or expansion have usually been generated by simple mechanical chart recorders (figs. 1 and 5). Movement of the instrument platform relative to the extensometer element causes a chart drum to rotate against a clock-driven pen. The commercially available water-stage recorders used by the USGS as compaction recorders require the addition of a second gear train to achieve a scale amplification of 10:1. In addition, the chart drums must be balanced, and the gear train adjusted for smoothest operation. An anti-backlash counterweight of about 5 grams, suspended from thread wrapped around the chart drum, is required to provide a constant minimal pre-load on the gear train. Resolution of the inked trace typically is $\pm$ 1 X 10^{-4} ft (0.03 mm), although specially modified instruments have been built that provide $\pm$ 2 X 10^{-5} (0.006 mm) ft resolution with 50:1 amplification. The recorder is positioned on the instrument platform so that the plane of rotation of the recorder drive pulley is parallel to the plane of oscillation of the counterweighted balance beam.

The recent development of reliable, moderately priced, battery-powered, multi-channel data loggers makes it feasible to collect aquifer deformation and related data directly in computer-compatible digital format. Thus it is possible to eliminate the tedious, expensive, and error-prone chores of manually picking data points from analog charts, tabulating and graphing the data, and keying them into a computer for analysis and modeling. A highly sensitive, linear, electrical analog of extensometer movement can be generated using a linear variable differential transformer (LVDT) as the displacement transducer. Pressure transducers sensing the heads in co-located piezometers can be monitored with the same data logger. Other parameters of interest, such as temperatures at and below the surface, barometric pressure, and precipitation can also be recorded. To conserve battery power the sensors can be switched on briefly at the desired sampling rate, perhaps once per hour, under data-logger control. Data are collected over a period of a month or more on magnetic tape or in solid-state memories for subsequent transfer to a computer.

If electronic sensing and recording are employed, it is essential to provide a supplementary direct-reading mechanical measuring system, such as a dial gauge, for calibration and to maintain data continuity in case of electronic failure or an off-scale condition resulting from excessive movement. The relatively limited dynamic range of transducer systems, compared to a mechanical recorder, may require fairly frequent, even monthly, rezeroing of the transducer during periods of rapid subsidence. Under these circumstances it is necessary to have a precise and convenient mechanical adjustment system that reliably preserves the fundamental instrument reference. A micrometer-adjustable transducer mounting is suitable (fig. 6A). Redundant mechanical and electronic recording can be linked very effectively and simply to provide the advantages of an immediately visible analog record as well as computer-compatible digital data.

Regardless of the type of extensometer and recording systems employed there should be afixed to the extensometer element two permanent reference marks whose distance from a well-defined measuring point on the instrument platform can be measured with an appropriate level of accuracy (at least ±0.1 mm), using a machinist's slide caliper, depth gage, or similar device. Continuing compaction eventually requires removing the upper mark and reattaching it below the (originally) lower mark. In this manner an uninterrupted, self-checking record of periodic compaction measurements is established independently of the recording systems.

Summary and Conclusions

The USGS experience with operating borehole extensometers in a wide range of circumstances may be generalized in the form of six fundamental requirements. Briefly stated these are:

1. The base of the extensometer system--that is, the bottom-hole anchor or subsurfance benchmark--must be stable with respect to the bottom of the hydrostratigraphic depth interval of interest.
2. The well casing must not disturb the system being measured by acting as a significant load-bearing structure within the compacting sedimentary column.
3. The instrument platform, which constitutes the extensometer datum at land surface, must be stable with respect to the top of the interval of interest.
4. The extensometer element (pipe or cable) must maintain an invariant length. A corollary of this requirement is that the borehole must be as straight and plumb as possible, to minimize downhole friction.
5. Above-ground mechanisms (counterweights, levers, pulleys, etc.) used to support the extensometer element must have negligible friction and must exert a constant uplift force on the element, regardless of movement of the land surface and instrument platform relative to the element.
6. The measuring and recording devices used to monitor displacement of the instrument datum (land surface) relative to the extensometer element must be linear, accurate, stable, and sensitive, and must not impose significant frictional or spring loads on the extensometer. They should also provide sufficient redundancy to insure that the record of cumulative displacement will not be lost during periods of routine instrument maintenance, equipment failure, or modification.

With careful design and construction these idealized requirements may be closely approximated; the resulting instruments have virtually no drift, very low noise, and strain resolutions that may exceed 1 part in 10^7. Aquifer-system deformations attributable to barometric pressure chang rainfall loading, and head changes of less than 10 mm can be resolved. Long-term records from standard extensometers, in conjunction with water-level piezometers, have made possible the determination of the aquifer-system properties that control land subsidence. Newer, extra-high resolution extensometers permit definition of the compressibility and hydraulic conductivity of thin, individual aquitards by means of short-term pumping tests.

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GROUND DEFORMATION IN THE GULF COAST REGION OF THE U.S. ASSOCIATED WITH FLUID WITHDRAWAL FROM A FIVE KILOMETER DEEP RESERVOIR

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Abstract

A 5 km deep, fault-bounded reservoir in the Gulf Coast area of the United States was tested as an unconventional source of natural gas. During the reservoir flow test, about 1.3 x 10^{+6} bbls of brine were withdrawn during an eight month period. The reservoir is located in an area of active natural subsidence. Ground deformation monitoring studies using first order levelling, tiltmeters of high sensitivity and microseismic monitoring were conducted to examine the potential for subsidence from such deep well withdrawal.

Results of first order levelling identify a pre-withdrawl character of surface elevation changes, a distinct reversal of this character during the withdrawal period and for about a year beyond that period, and a subsequent return to a pre-withdrawal character. Tiltmeters also suggest a change in surface deformation character during the period of withdrawal. An increase in the rate of occurrence of very small microseisms located about the reservoir area was also observed. The occurrence of microseismic events, and short-term tilt events for which tilt is not recovered, suggest that deformation occurring is not only an elastic response to fluid withdrawal from the reservoir, but also represents permanent deformation.

Introduction

Deep reservoirs (5 km) in the Gulf Coast region of the United States are being tested as a potential source of methane gas. The reservoirs were formed as a result of faulting that juxtaposed fluid rich sand layers and clay shales. The increased overburden above the downdropped blocks, and the lack of an efficient pathway by which reservoir fluids can escape, combine to produce relatively high fluid temperatures and pressures within the reservoir. The reservoirs are of interest because of the methane gas contained in the geothermal/geopressured fluid.

Ground deformation studies, co-sponsored by the Gas Research Institute and the U.S. Department of Energy, have been conducted coincident with testing of these reservoirs. During reservoir testing large volumes of brine were removed from depths of about 5 km. The dissolved gases were removed from the brine, and then the waste fluids were reinjected into unbounded sands at a depth of 2 to 3 km. Alternatively, the waste fluids are disposed of offshore. Ground deformation studies were designed to monitor and document any deformation that may result from the fluid withdrawal and reinjection.

At Sweet Lake, Louisiana a reservoir flow test was carried out between late June 1981 and February 1982. Flow rates ranged from about 1 x 10^{+5} to 3 x 10^{+5} barrels per month. During the test, more than one million barrels of fluid were withdrawn. Since the flow test was terminated only minor volumes of fluid have been withdrawn from the reservoir. Evidence of ground deformation, during and after the flow test, as recorded by seismometers, tiltmeters and a levelling array, is presented in this paper.

Approach

A plan was developed to study ground deformation on several time and sensitivity scales. Dynamic deformation, occurring on the order of seconds, was monitored by a telemetered array of short-period seismometers. Longer period strain events, with durations of minutes to days, were monitored by various types of tiltmeters. Long term changes in elevation were measured by monthly first-order levelling of a 2 km array. In this manner, background deformation, and any deformation associated with reservoir testing, would be detected whether it occurred on a time scale of seconds to months.

Microseismic Studies - Microseismicity in the vicinity of the well was monitored by an array of eight short period vertical seismometers. These instruments were installed ten to thirty meters below the ground surface over an elliptical area of about 10 by 15 km around the well (Figure 1). The network began operation about a year prior to the reservoir test to monitor background seismicity.

Microseismic events of estimated magnitude -1 to +1 were detected very infrequently prior to flow testing of the reservoir. The frequency and number of detected events increased dramatically upon commencement of withdrawal activities. The recorded seismograms have an emergent character with no clear body-wave arrivals. This character is probably caused by the unconsolidated sediments in which the events originate, and through which seismic energy must travel. The observed arrivals travel at a speed thought to be appropriate for surface waves. The generation and propagation of these arrivals are not yet completely understood.

The temporal distribution of microseismicity ranges from bursts of events closely clustered in time (minutes to hours) to single events separated by days to tens of days. Microseismic events are spatially distributed in an apparently random manner about the well and subsurface reservoir. Spatial trends that can be identified are highly subjective and must be viewed with caution. The precision with which the detected microseisms can be located, precludes identification of specific structures that are seismically active.

Tiltmeter Studies - Different types of tiltmeters were installed to examine their reliability in the hostile environment of the Gulf Coast region (Figure 1). This environment is characterized by temperatures that range from freezing to over 50°C within instrument enclosures; rainfall that regularly produces onsite flooding; and effects of seasonal storms and hurricanes that are severe. Only a bubble-type tiltmeter, located along the north-south leg of the levelling array, produced a data set of any coherence. Measurement of tilt began just prior to the initiation of reservoir testing activities; thus, no background data were obtained.

A typical tilt excursion occurs over a period of hours to days and has an amplitude of several microradians (Figure 2). More of these events are time coincident with onsite rainfall. A few tilt events, however, are observed independent of any meteorological phenomena. A speculative explanation for these observations is that surface flooding triggers tilt events that would eventually occur in the absence of such triggers.

Longer term tilt trends suggest that the local ground surface is gradually tilting southward (Figure 2). These data are consistent with the natural subsidence that is observed on a broader scale for the Gulf Coast region as a whole (Trahan, 1982).

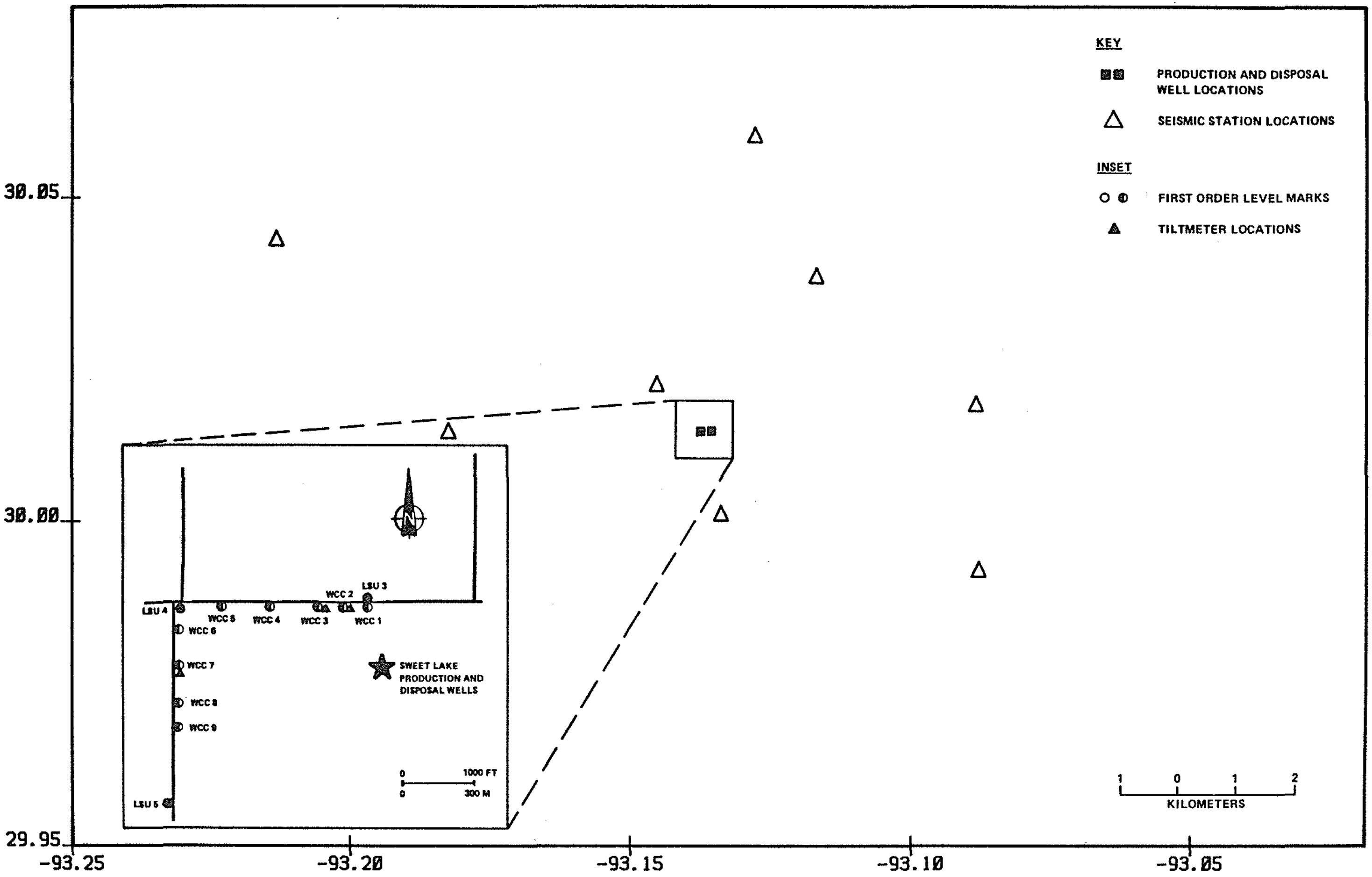
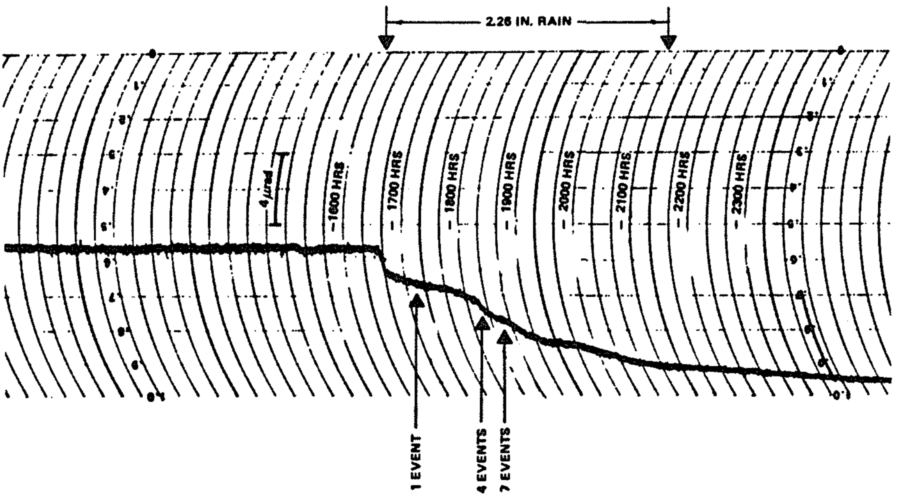


FIGURE 1.—LOCATION MAP OF INSTRUMENTATION ARRAY



TILT EXCURSION SHOWING COINCIDENT MICROSEISMICITY

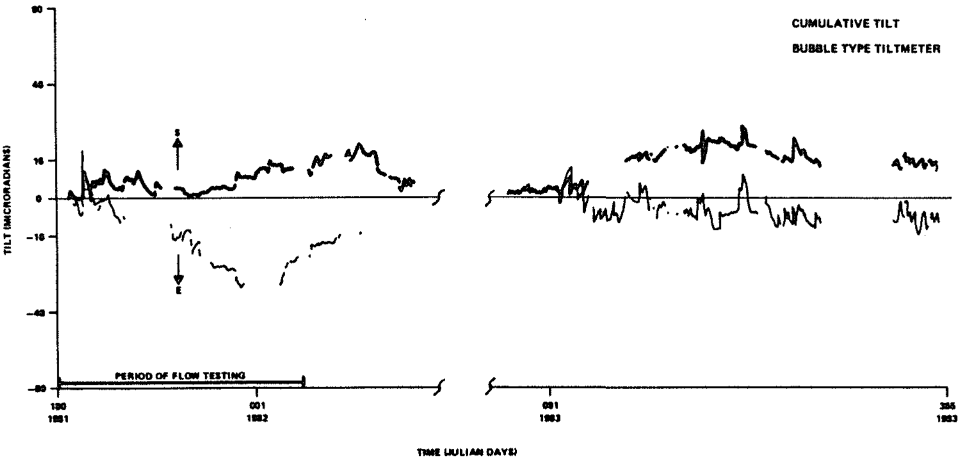


FIGURE 2.-- TILT MEASURED 1 KM WEST OF WELL
FOR JUNE 1981-JUNE 1982 AND JANUARY 1983-JANUARY 1984

<u>First Order Levelling Studies</u> - First order levelling studies were initiated about six months prior to reservoir testing; the complete array was established about three months prior to well flow activity. The array consists of ten first order level marks sited in a 2 km long inverted "L" pattern adjacent to the well (Figure 1). Marks were located with a spacing of 100 to 300 m. The east-west leg of the array begins about 250 m north of the well and extends about 1 km to the west. The north-south leg begins about

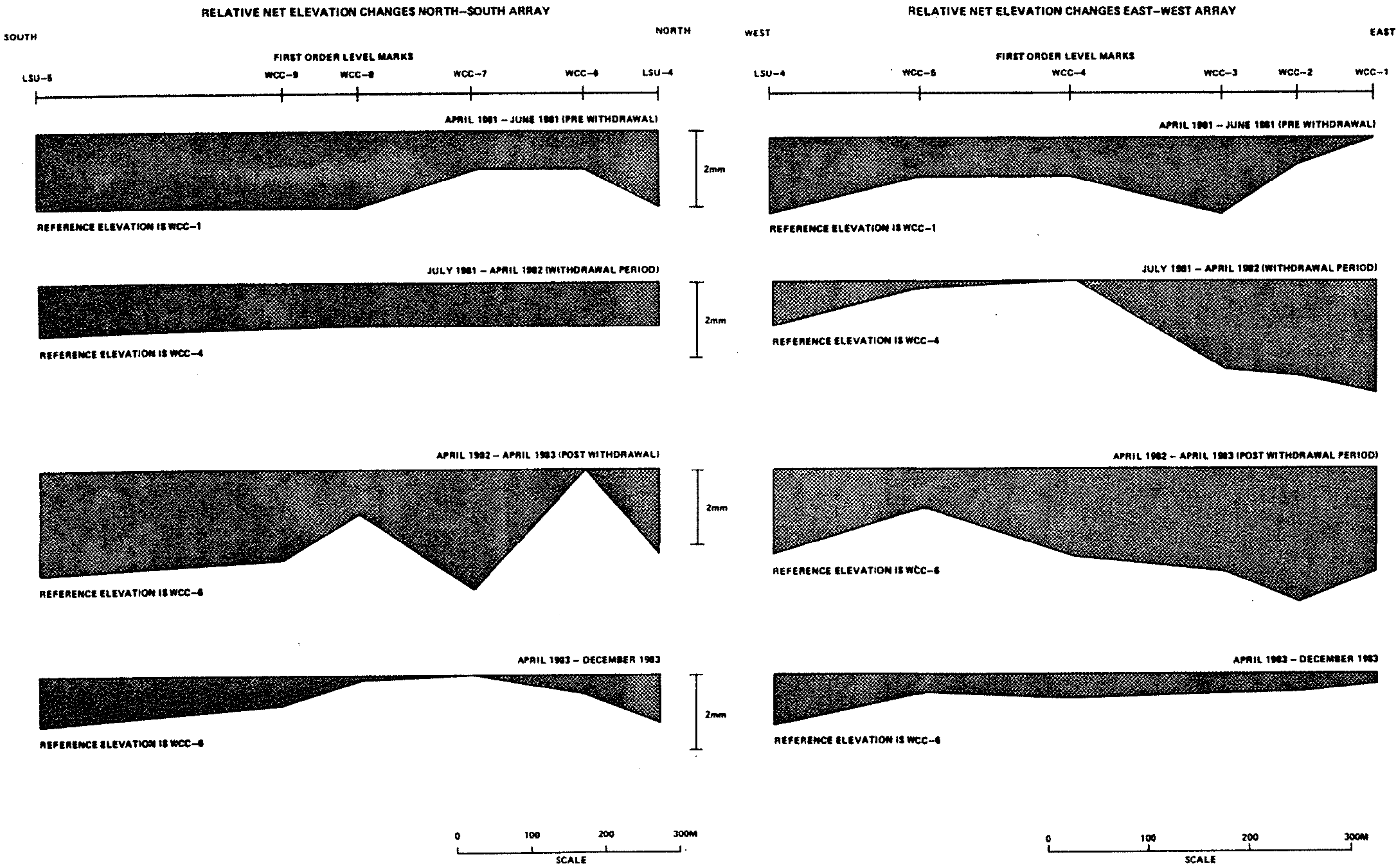


FIGURE 3.—RELATIVE NET ELEVATION CHANGES

1 km west of the well and is also about 1 km long. It terminates southwest of the well. The array was relevelled on a monthly basis to define in detail elevation changes associated with reservoir testing.

Along the east-west leg of the array, relative elevation changes indicate a general down to the west trend prior to fluid withdrawal activities (Figure 3). This observation is based on measurements taken over six months for the end points of the leg, and over three months for the complete array. The changes observed, however, are within the range of expected reading errors with respect to absolute elevations, and hence conclusions based on the observations must be considered tentative. Nevertheless, the observed trend is consistent from month to month.

After the onset of fluid withdrawal, and continuing for more than a year beyond the end of the flow test, the trend of relative elevation changes for the east-west leg of the array appears to have reversed in direction (Figure 3). Along most of the leg, relative elevation changes trend down to the east (toward the well) at a rate of about 2 mm/yr. The highest point along the leg is observed about 300 m from its western end. This point has been arbitrarily held at zero elevation change for reference, but may itself be subsiding. When the trend of relative elevation based on changes observed prior to withdrawal are taken into account (about 2 mm/yr towards the west)the inferred total relative change is 5 mm/yr.

Along the north-south leg of the array relative elevation changes for the two months prior to fluid withdrawal trend down to the south at 1 to 2 mm over the kilometer long leg. This observation is compatible with the regional subsidence vector which is directed southwesterly at 2 mm/km or more (Trahan, 1982).

Beginning with the onset of fluid withdrawal the down to the south trend is slowed or stopped (Figure 3). During the 18 months after fluid withdrawal ceased, the down to the south relative change in elevation is re-established, but the trend is not as uniform as before.

Note again that the summary of relative elevations (Figure 3) assumes no uplift has occurred. Under this assumption, elevation changes on each leg, are shown relative to the array mark with the smallest negative elevation change. This mark is noted on the figure. Given that the whole system is likely subsiding within a regional framework, actual elevation changes are probably greater by some constant value than those shown.

Discussion

The trends of relative changes in elevation determined from first-order levelling data exhibit a pattern that is consistent with the trends expected in response to fluid withdrawal. When the down to the west trend observed prior to the reservoir flow test is taken into account, relative elevation changes of up to 5 mm trending down to the east (toward the well) are derived. These values are substantial considering the small volume of fluid withdrawn relative to the total reservoir volume. Thus, observed elevation changes may represent a direct and rapid response to fluid volume changes at depth. The trend of relative elevation changes returns to pre-test patterns about one year after withdrawal activities ceased.

Levelling data do not reach far enough beyond the well area to determine whether observed changes represent a permanent or an elastic response to fluid withdrawal. Microearthquakes that occur in the immediate vicinity of the reservoir,however, suggest that at least some permanent deformation has occurred. Dynamic displacements causing the microseismic events represent permanent deformation. The percentage of permanent deformation that is accompanied by dynamic (seismic) displacement cannot be estimated. Observed geologic displacements within the Gulf Coast region, and the relatively aseismic history of this area, suggest that most ongoing deformation in the region occurs aseismically.

Tilt data suggest that deformation affecting the near surface includes both permanent and elastic components. Heavy rainfall that produces surface flooding is strongly correlated with tilt events. In some cases, following the subsequent runoff of surface water, the observed tilt almost completely recovers to pre-flooding values. For other events, however, recovery is either incomplete or negligible, indicating that surface flooding can trigger permanent surface deformation. Tilt events are also infrequently observed in the absence of rainfall and surface flooding. This leads to the conclusion that deformation is ongoing, and only triggered by surface flooding as opposed to being caused only by such events.

A moderate correlation exists between the occurrence of microseismicity and tilt events. Both tilt and seismic events are also observed independent of each other. Thus, there does not seem to be a direct cause and effect between them.

Conclusions

Changes in ground surface elevations are interpreted to be caused by fluid withdrawal from a deep confined reservoir. The changes occur shortly after fluid withdrawal begins. This conclusion is based on observations at one 5 km deep well in the Gulf Coast region of the United States. The effect of withdrawal continues for about a year beyond the termination of withdrawal activities. Surface elevation changes are interpreted to trend down towards the well and may be as large as 5 mm/yr or more. This value is determined for an array whose closest point is about 300 m north of the well.

Observations of coincident microseismicity and tilt suggest that at least some of the observed deformation is occurring non-elastically. Discrete tilt events and microseismicity occur at an increased rate during and following the reservoir flow test. These observations suggest that deformation occurs at least partially in discrete episodes rather than in a slow uniform manner.

Observed elevation changes approximate in a general way the shape expected from a compacting reservoir. That is, changes measured at the surface increase in magnitude towards the well, consistent with the shape of a classic subsidence bowl. Seismicity and discrete tilt excursions suggest that at least some of the deformation may be more representative of block shifting or segmented displacement events. It is concluded that the reservoir monitored in this study may be deforming in both styles.

A second reservoir, at Rockefeller Refuge about 40 km southeast of the Sweet Lake site, is currently being tested. Ground deformation studies are also being conducted at that site. If similar results are obtained, the speculative hypotheses presented here will be more strongly supported.

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REVIEW OF TECHNIQUES FOR DETERMINING VERTICAL GROUND MOVEMENTS FROM LEVELLING DATA

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Abstract

A review of existing mathematical models to obtain the spatial and temporal representation of vertical ground movements from relevellings is presented. Models are classified in terms of the following characteristics:

i) Model design (displacement or velocity profiles, areal modelling of displacements or velocities),
ii) spatial data distribution (scattered relevelled segments or relevelled networks) and
iii) temporal data distribution (data from two or several surveys).

Solutions to the vertical indeterminacy problem of different models are analysed. Particular attention is paid to the spatial and temporal correlation of levelling data and its effect on computed results. Most suitable mathematical models for different types of studies are recommended.

Introduction

A mathematical model is a functional relationship between unknown parameters and observed quantities. This is a review of existing models designed to detect and represent vertical ground movements from levelling data.

Two observable quantities are the input in the mathematical models: vertical displacements at one point, from sea level records, u, and rigorous height-difference differences between two points, from levelling and gravity or differences of lake level trends, $\delta\Delta H$. Former reviews can be found elsewhere on the state of the art of geodetic levelling (Vaníček et al., 1980), and other kinds of data such as hydrostatic levelling (Sneddon, 1979), sea level trend analysis (Lennon, 1978; Vaníček, 1978), lake level trend analysis (Coordinating Committee on Great Lakes, 1977), gravity differences (Torge, 1982) and absolute gravity (Cannizo et al., 1978; Faller et al., 1980).

For the definition of terms and notation the reader is referred to Vaníček and Krakiwsky (1982).

Classification of Techniques

The mathematical models can be classified according to their design: they can provide either vertical displacement profiles or areal representations.

Mathematical models are also characterized by their data:

i) Vertical ground movements are said to be absolute if they use the geoid as a reference surface or relative if any other reference surface is used.

ii) In space, the data may be distributed in the form of either disconnected segments or in the form of connected segments forming a levelling network.

iii) In time, the data may be distributed either in the form of two or more surveys made at distinct epochs or, alternatively, in the form of surveys randomly scattered in time.

Detection of Vertical Ground Movements from Disconnected Segments

A relevelled segment between two benchmarks P_i and P_j at epochs τ_1 and τ_2 provides a height-difference difference

$$\delta\Delta H_{ij}(\tau_1,\tau_2) = \Delta H_{ij}(\tau_2) - \Delta H_{ij}(\tau_1). \tag{1}$$

Height-difference differences can be portrayed as a spatial profile of relative height changes.

A set of disconnected segments or spatial profiles may be 'held together' by means of a surface regression. They provide an areal representation of vertical displacements.

$$u(x,y) = \underline{\Phi}^T \underline{c}_u \; . \tag{2}$$

The least squares solution of the coefficients reads

$$\underline{\hat{c}}_u = (\underline{\Delta\Phi}\; \underline{C}_{\delta\Delta H}^{-1}\; \underline{\Delta\Phi}^T)^{-1} \underline{\Delta\Phi}\; \underline{C}_{\delta\Delta H}^{-1}\; \underline{\delta\Delta H} \; , \tag{3}$$

and the covariance matrix of the surface is

$$\underline{C}_u = \underline{\Phi}^T \underline{C}_{\hat{c}_u} \underline{\Phi} \quad , \tag{4}$$

where

$$\underline{C}_{\hat{c}_u} = (\underline{\Delta\Phi}\; \underline{C}_{\delta\Delta H}^{-1}\; \underline{\Delta\Phi}^T)^{-1} \; . \tag{5}$$

Here $\underline{\hat{c}}_u$ is the least squares estimate of the base function coefficients, $\underline{\Phi}$ and $\underline{\Delta\Phi}$ are vectors of space base functions and their differences, $\underline{\delta\Delta H}$ is the vector of all height-difference differences and $\underline{C}_{\delta\Delta H}$ and $\underline{C}_{\hat{c}_u}$ are the covariance matrices of the height-difference differences and the estimated coefficients.

The same mathematical apparatus as used in eqns. (3) to (5) is also clearly applicable if an areal representation of constant vertical velocities $\dot{u}(x,y)$ is sought. The velocity of change of $\delta\Delta H$ is given by

$$\delta\dot{\Delta}H_{ij} = \delta\Delta H_{ij}/(\tau_2-\tau_1) \; . \tag{6}$$

If more than two levellings are available a vertical displacement history over an area can be constructed, i.e.

$$u(x,y,\tau) = \underline{v}^T \underline{\Phi}(x,y) \underline{T}^T(\tau)\; \underline{\tilde{c}}\; \underline{v} \; . \tag{7}$$

All pertinent equations of its least squares solution read

$$\underline{c} = (\underline{A}\ \underline{C}^{-1}_{\delta\Delta H}\ \underline{A}^{T})^{-1}\ \underline{A}\ \underline{C}^{-1}_{\delta\Delta H}\ \underline{\delta\Delta H}\ , \tag{8}$$

where

$$\underline{A}^{T} = \underline{v}^{T}\ \underline{\Delta\Phi}\ \underline{\Delta T}^{T}\ , \tag{9}$$

and

$$\underline{c} = \underline{\tilde{c}}\ \underline{v}\ , \tag{10}$$

where $\underline{T}^{T}$ and ΔT^{T} are transposed vectors of time base functions and their differences, $\underline{\tilde{c}}$ is a matrix of space-time coefficients and $\underline{v}$ is a vector of ones.

The four-dimensional representation given by eqn. (7) can be used as a tool to obtain a <u>temporal profile history of vertical displacements</u> $u(x_k,y_k,\tau)$ at any given point P_k.

<u>Detection of Vertical Ground Movements from Connected Segments</u>

Connected segments allow to perform a simultaneous multivariate analysis of all levelling data and their accuracies. This procedure is referred to as an adjustment in geodesy or as an inversion in geophysics.

The mathematical model for the detection of vertical ground movements in a twice levelled net is expressed as

$$\underline{A}\ \underline{u} = \underline{\delta\Delta H}\ , \tag{11}$$

where $\underline{A}$ is the design matrix of the network, $\underline{u}$ is a vector of vertical displacements and $\underline{\delta\Delta H}$ is the vector of observed height difference differences. All pertinent equations in the least squares solution of the vertical displacements read

$$\underline{\hat{u}} = (\underline{A}^{T}\ \underline{C}^{-1}_{\delta\Delta H}\underline{A})^{-1}\ \underline{A}^{T}\ \underline{C}^{-1}_{\delta\Delta H}\ \underline{\delta\Delta H}\ , \tag{12}$$

$$\underline{\delta\hat{\Delta}H} = \underline{A}\ \underline{\hat{u}}\ , \tag{13}$$

$$\underline{\hat{r}} = \underline{\delta\hat{\Delta}H} - \underline{\delta\Delta H}\ , \tag{14}$$

$$\hat{\sigma}^{2}_{0} = (\underline{\hat{r}}^{T}\underline{C}^{-1}_{\delta\Delta H}\ \underline{\hat{r}})/(m-n)\ , \tag{15}$$

$$\underline{C}_{\hat{u}} = (\underline{A}^{T}\ \underline{C}^{-1}_{\delta\Delta H}\ \underline{A})^{-1}\ , \tag{16}$$

where $\underline{\hat{u}}$, $\underline{\hat{r}}$ and $\underline{\delta\hat{\Delta}H}$ are the least squares estimates of the displacements, the residuals and the height difference differences. $\underline{C}_{\delta\Delta H}$ and $\underline{C}_{\hat{u}}$ are the covariance matrics of the observed height differences of the estimated vertical displacements, $\hat{\sigma}^{2}_{0}$ is the a posteriori variance factor and m and n are the number of observations and displacements respectively.

The same apparatus is clearly applicable also if constant vertical velocities $\underline{\dot{u}}$ are sought, i.e.,

$$\underline{A}\ \underline{\dot{u}} = \underline{\delta\dot{\Delta}H}\ \ , \tag{17}$$

where

$$\underline{\dot{u}} = (\underline{H}(\tau_2)-\underline{H}(\tau_1))/(\tau_2-\tau_1) \ , \qquad (18)$$

and

$$\underline{\delta\dot{\Delta}H} = (\underline{\Delta H}(\tau_2)-\underline{\Delta H}(\tau_1))/(\tau_2-\tau_1) \ . \qquad (19)$$

If more than two levellings are available information of a higher order in time can be extracted.

Equations (11) and (17) have been formulated as explicit models in the observations. They can be also formulated in terms of condition models

$$\underline{B} \ \underline{\delta\Delta H} = \underline{0} \ , \qquad (20)$$

or

$$\underline{B} \ \underline{\delta\dot{\Delta}H} = \underline{0} \ , \qquad (21)$$

or as linear implicit models

$$\underline{A} \ \underline{u} + \underline{B} \ \underline{\delta\Delta H} = \underline{0} \ , \qquad (22)$$

or

$$\underline{A} \ \underline{\dot{u}} + \underline{B} \ \underline{\delta\dot{\Delta}H} = \underline{0} \ . \qquad (23)$$

All lead to equivalent vertical displacements or velocities respectively.

The displacement and constant velocity models for connected segments provide displacement or velocity profiles at relevelled points of a network. If an areal representation is further required, a surface regression using eqns. (3) to (5) over the estimated uplifts at relevelled points can be attempted.

An alternative approach is a two component adjustment with a prediction of the signal or least squares collocation (Moritz, 1978), i.e.,

$$\underline{A} \ \underline{u} + \underline{B} \ (\underline{s} + \underline{n}) = \underline{0} \ . \qquad (24)$$

The signal predicted by this model is given by

$$\underline{u}_p = - \underline{C}_{s_p s} \ \underline{C}_r^{-1} \ \underline{\delta\Delta H} \qquad (25)$$

where $\underline{C}_{s_p s}$ is the cross-covariance matrix of the signal and

$\underline{C}_r$ is given by

$$\underline{C}_r = \underline{C}_s + \underline{C}_n \qquad (26)$$

where $\underline{C}_s$ and $\underline{C}_n$ are the covariance matrices of the signal and noise respectively.

The Indeterminacy Problem

All the above models show a vertical indeterminacy because only height difference differences or their time derivatives have been included. The systems of normal equations of all models have a rank deficiency of one, i.e., they are singular. There are two approaches to solve this problem:

imposing actual constraints or, alternatively, introducing artificial constraints.

If the first approach is followed, the models are reformulated and further information (sea level trends) is added. The computed vertical displacements are absolute, i.e., they are referred to the geoid.

When the second approach is advocated, any constrained solution avoids the singularity in the models. However, only a minimum constraint,i.e., fixing only one point, frees the solution of as many unnecessary physical hypothesis as it is possible. Mathematically, a minimum constraint is the minimum number of conditions to be imposed to make the ranks of the design matrix and of the system of normal equations equal to the number of unknowns. All vertical displacements and their accuracies found through this approach are relative to the point fixed. Alternatively, the use of generalized inverses leads to a large number of choices. The Moore-Penrose inverse is usually selected. The values of $\underline{u}$ are relative to the centroid of the network.

The Covariance Matrix of the Height Difference Differences

$\underline{C}_{\delta\Delta H}$ in equations (3), (5), (8), (12) and also in (26) ($\underline{C}_n = \underline{C}_{\delta\Delta H}$) rigorously reads

$$\underline{C}_{\delta\Delta H} = \underline{C}_{\Delta H(\tau_1)} - 2\,\underline{C}_{\Delta H(\tau_1)\Delta H(\tau_2)} + \underline{C}_{\Delta H(\tau_2)} \;, \qquad (27)$$

where $\underline{C}_{\Delta H(\tau_1)}$ and $\underline{C}_{\Delta H(\tau_2)}$ are the covariance matrices of the height differences observed at epochs τ_1 and τ_2 respectively, and $\underline{C}_{\Delta H(\tau_1)\Delta H(\tau_2)}$ is the temporal cross-covariance matrix of the observations between both epochs. However, it is not feasible at present to rigorously assess the statistical dependence of the observations in space, i.e., the statistical dependence of one observation with respect to the rest at a given epoch. Neither is it possible to evaluate the statistical dependence in time of each of the observations made at one epoch with respect to all the observations made at another epoch. This results in neglecting all off-diagonal elements of $\underline{C}_{\Delta H(\tau_1)}$ and $\underline{C}_{\Delta H(\tau_2)}$, and the entire $\underline{C}_{\Delta H(\tau_1)\Delta H(\tau_2)}$. By necessity, therefore, all models assume total statistical independence in space and time.

Clearly, by neglecting $\underline{C}_{\Delta H(\tau_1)\Delta H(\tau_2)}$, for positive temporal cross-correlations, an underestimation of the accuracy of the computed displacements is made in all models.

Prediction Power of the Models

A comparative analysis of the different models can be performed in terms of their prediction power in space and time.

A spatial profile of relative height changes provides a linear interpolation between two epochs only at adjacent relevelled benchmarks (Mark, et al, 1981; Torge and Kanngieser, 1980). It does not make use of any possible redundancies.

An areal representation of vertical displacements, or constant velocities, provides a continuous, smooth, linear in time characterization. It can use connected or disconnected segments and is dependent upon their distribution in space. A vertical displacement history over an area is a four-dimensional generalization of the above model. Both areal representations provide a posteriori error estimators. *The mathematical model for the detection of vertical ground movements from* disconnected segments has been used extensively. Vaníček and Nagy (1979, 1981) have obtained displacement surfaces. Vaníček and Christodulidis (1974) and Vaníček (1975, 1976) have obtained velocity surfaces. A temporal displacement history, described by eqn. (7) has been attempted in the area of southern California (Vaníček, et al, 1979).

A least squares estimation of linear displacements or constant velocities from connected segments, a levelling net adjustment, provides kinematic information at relevelled benchmarks. Use of all existing redundancies in the observations is made. Rigorous error estimates are available. Explicit models in the observations have been used, for example, by Gale (1970), Vaníček and Hamilton (1972) and Holdahl and Morrison (1974). Holdahl (1977, 1982) has further attempted a surface regession over the estimated displacements. Condition models have been used by Andersen and Remmer (1982) and Suutarinen (1983).

A two component adjustment with a prediction of the signal of a levelling net provides actual and predicted vertical displacements, or constant velocities, at junction benchmarks and selected points respectively. An autocovariance function has to be postulated which dictates the spatial prediction performance. All redundancies in the observations are used to estimate a posteriori errors. Least squares collocation has been attempted, for example, by Hein (1980), Hein and Kistermann (1981), Torge (1981) and Kanngieser (1982).

In this review, solutions have been presented based on the use of normal equations. Clearly, other approaches like singular value decompostion or Householder transformations (Lawson and Hanson, 1974) are also applicable.

Recommendations

The selection of a mathematical model for a particular task depends upon the desired amount of kinematic information to be extracted. It is a function of the desired prediction power in space and time.

Spatial levelling profiles are usually designed as a diagnosis tool of vertical ground movements. They are the least expensive in terms of data collection.

Areal representations are useful when smooth characterizations in space and time are required. Special care is required to recognize a priori spatial discontinuities. Areal representations from disconnected segments are the least expensive surface model. They can use relevelling data which results from the maintenance of existing geodetic networks.
Areal representations from connected segments, on the other hand, require a costlier investment but higher spatial resolution should be expected.

Least squares kinematic adjustments of levelling nets are the most accurate approach of detecting vertical ground movements. The high cost of their planning and data collection usually constrain them to local areas. Kinematic studies in large nets can often be pursued only with long term national support.

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AN EXPERIMENTAL INVESTIGATION ON THE STABILITY OF LEVELING BENCH MARKS INSERTED INTO SEDIMENTARY SOILS

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Abstract

The results of an experimental investigation on the stability of leveling bench marks into sedimentary soils are analyzed.

The use of a new type of rod-like bench mark anchored at a depth of 5-6 meters allowed to distinguish and individuate accidental movements due to local causes from those of a more general nature due to deeper causes. Vibrations, mechanical and hydrostatic stresses, and oxidation processes can in fact negatively influence the stability of reference points and mask the actual displacements of land surface.

Some interesting, recurring up and down displacements of land surface, involving almost all the bench marks under study and linked to water table variations are illustrated. Considerations of geotechnique on soil mechanic properties are used to interpret the phenomenon and its importance.

Importance of Type and Location of Bench Marks in Measuring the Subsidence in Sedimentary Soils

In high precision geometric leveling for measuring subsidence, bench marks (BM) for which the relative elevation and their temporal variations are determined, have a great importance. They are required to be permanently established in ground and their altimetrical movements must reflect the total land sinking. Therefore they must not be subjected to disturbances of local altimetrical movements which are not related to the phenomenon under study. Local disturbances are usually superficial, therefore setting up bench marks in the upper most strata is not always feasible because of the multi-natured influences, such as human, meteorological and biological, on surface soils.

With the financial support from the "Commissione Geodetica Italiana"(CGI) and the collaboration of CNR-Laboratorio Studio Dinamica Grandi Masse of Venice, the Direzione Generale del Catasto of Rome, and the Istituto di Topografia, Fotogrammetria e Geofisica del Politecnico of Milan, the testing of the behavior of 59 bench marks which were placed by the Consonda S.p.A., Milan, was begun in 1976 using rod-type bench marks anchored into depths up to about 6 meters (Fig. 1).

These rod-type bench marks were placed near to as many traditional bench marks (superficial) of line no. 16 of the national leveling network IGM, between Cervia and Portomaggiore, and established into alluvial soil, subject to subsidence phenomena. The experiment consisted in measuring according to high specification, every six months, the elevation differences between two adjacent bench marks that from now on we will simply call: IGM bench marks and CGI bench marks.

The anchorage depth of rods was determined to be 4-5 meters, based on the range of fluctuations of the water table. The water table variation is obtained at the same time as the elevations differences measured by the water table measures in wells located near the CGI bench marks.

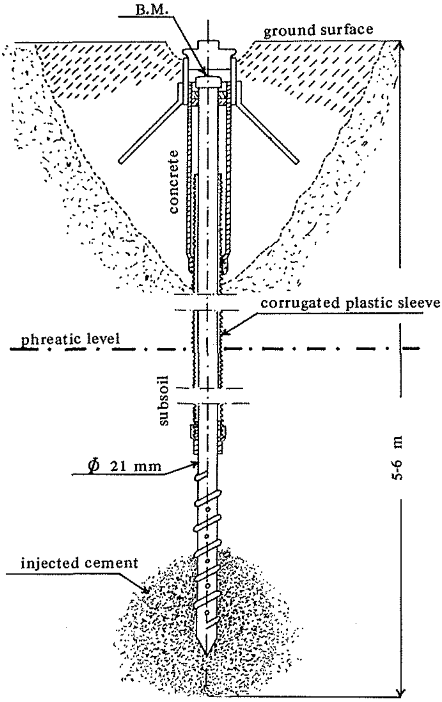

Fig. 1. Rod-type bench mark.

Measurement of elevation differences between IGM and CGI bench marks was made by high precision automatic levels, and was repeated successively at least twice. The latter were reference points because they were anchored more deeply into the soil. A comparison between the results of the repeated measures was a control of the precision; the mean square error of the mean elevation differences has always been less than 0.1 mm. The results of these measures, repeated every spring and autumn over a 6-year period (13 on the whole) have been graphically represented. Figure 2 reports the elevation differences as recorded together with the water table variations in two very significant cases.

Even from a simple qualitative observation of these graphs, the close correlation existing between phreatic level and variation of elevation differences is quite clear. Besides seasonal influences on the water table, other factors occasionally exert an influence thus causing a change in the graph on variations in elevation differences.

The study of this correlation in relation to its magnitude and to the lithological nature of the soil included between the surface where IGM bench marks are established and the depth where CGI bench marks are anchored will be dealt with in this paper.

For this reason detailed core samples of the soil where the more significant bench marks are located were taken. An accurate geotechnical investigation was made. At these bench marks from 1980 on, elevation differences and water table fluctuations were measured each month instead of every six months.

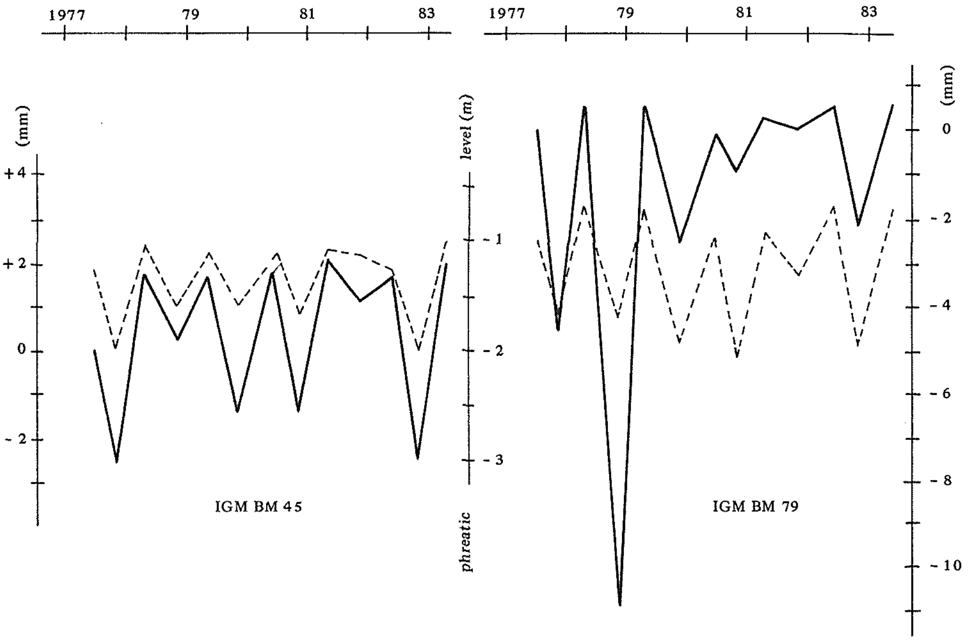

Fig. 2. Solid line indicates changes in elevation differences of IGM BM with repsect to CGI BM (mm). Dotted line is water-bearing surface oscillation with respect to ground surface (m).

The first part of our work refers to the investigation of the BM 45 and 79. The last part extends the results in order to interpret the other bench marks.

Data Analyis of Bench Marks 45 and 79

The first step in analyzing the results was to select the measures, correlated in time, of elevation differences between IGM BM and CGI BM not due to seasonal hydrological factors. This was done by interpolating a straight line which simulates the mean trend. The choosen values with respect to this straight line were analyzed in relation to the depth of the unconfined groundwater with respect to ground surface for each measure. The trend of this relationship can be interpretated as being linear and the parameters of the interpolator line are obtained with the least squares techniques. Figure 3 shows the two lines each with its own characteristic slope and confidence bands.

Soil properties at the site of the surface IGM bench mark and the anchored CGI bench mark was tested as previously mentioned by continual core samples and measures from static penetrometers driven into the ground down to a depth of 50 m. The Istituto di Costruzioni Marittime e di Geotecnica of the University of Padua carried out the study of the soil mechanics and supplied the necessary parameters for the investigation.

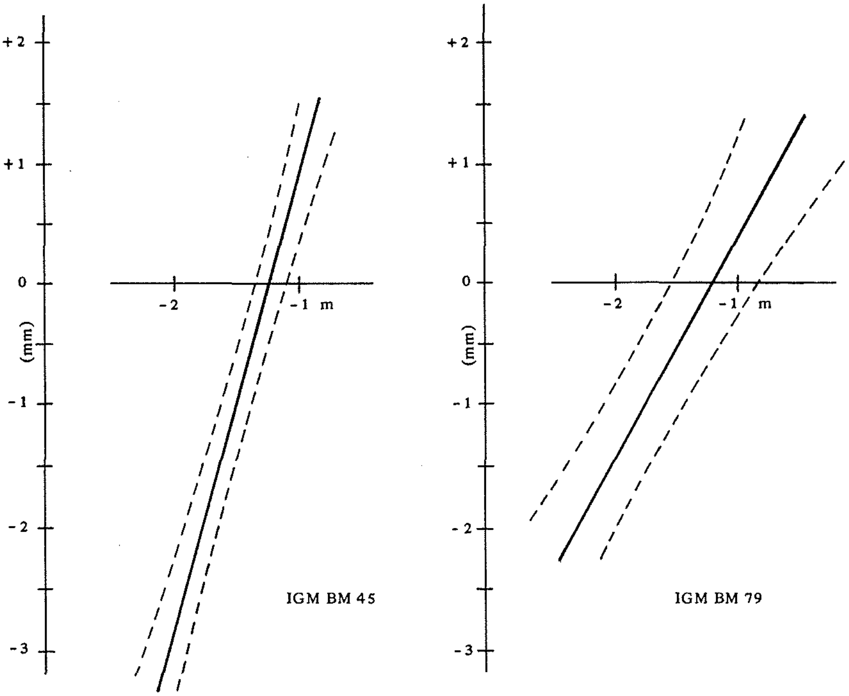

Fig. 3. Correlation between elevation changes (IGM-CGI bench marks) and water table fluctuations of bench marks 45 and 79.

Soil Mechanic Aspects

The fluctuations, within soil layers, of the free surface (i.e., water surface on which the atmospheric pressure acts) induce variations in the stress state of the subsoil. Consequently there is a build up of deformations which are proportional to the mechanical characteristics of the considered soil.

It is reasonable to assume that, over a limited period of time, the variations of the phreatic surface, should be bounded within levels which occurred in the past during the previous fluctuations. Furthermore, in these circumstances, the problem may be considered as being one-dimensional since, on flat land, the free surface fluctuations occur almost simultaneously over great horizontal extensions. It follows that the compression or the heave of the soil layers happens mainly in the vertical directions, i.e. the horizontal deformation $\varepsilon_h=0$.

The compressibility modulus of soil is then determined in the laboratory with oedometric tests which reproduce the deformation-boundary conditions given above. The experimental results, of these confined compression-tests, give the well-known strain-stress curves of the type reproduced in figure 4. The loop A-B-C represents the behavior of an over-consolidated soil; the vertical consolidation pressure ensures that all points on the loop have tensions lower than the stress present at point A.

This model of over-consolidation seems to give a good representation of the evolution of tensions within soils. Such evolutions always occur inside an over-consolidation loop of the type A-B-C; where A-B is an unloading and B-C is a loading stage. Anyoneloop can be defined by its mean slope, called the recompression index (RR):

(1) $$RR = \Delta\mathcal{E} / \Delta \log \sigma'_v$$

where

$\Delta\mathcal{E}$ difference of vertical deformation

$\Delta \log \sigma'_v$ logarithmic difference of vertical effective stress.

The settlement or heave of a thin layer of homogeneous soil, characterized by a recompression index RR, is evaluated by the expression

(2) $$\Delta H = RR\ H \log(\sigma'_v + \Delta\sigma'_v / \sigma'_v)$$

If thick layers or heterogeneous formations are present, equation (2) is considered as a sum of terms

(3) $$s = \Sigma\Delta H = \Sigma RR\, H \log(\sigma'_v + \Delta\sigma'_v / \sigma'_v)$$

In the equations (2) and (3), H represents the thickness of the considered layer, σ'_v and $\Delta\sigma'_v$ are the effective vertical stress and the net increase (or decrease) in vertical effective stress respectively, being that the stresses are evaluated in the mid-point of the layer. The granular formations are assumed as uncompressible when alternate layers of sandy and clayey soils are present in a site.

In cohesive soil of medium and low plasticity, the recompression ratio usually assumes values from 0.005 to 0.05 and must be defined experimentally.

The vertical effective stress is evaluated with the equation

(4) $$\sigma'_v = \Sigma\gamma H$$

where γ is the submerged unit weight for the soil located under the free surface. The net variation in vertical effective stress due to the maximum measured fall (or rise) of the free surface is given, below the level Z_{max} (maximum depth of the free surface), by the following equation which refers to the symbols in figure 5

(5) $$\Delta\sigma'_{v_{max}} = \int_{o}^{Z_{max}} \gamma_n \cdot dZ - \left[\int_{o}^{Z_{min}} \gamma_n \cdot dZ + \int_{Z_{min}}^{Z_{max}} \gamma' \, dZ \right] =$$

$$= \int_{Z_{min}}^{Z_{max}} (\gamma n - \gamma') \, dZ$$

Within the depth where fluctuations of the free surface occur, the average increase in vertical effective stress is given by

(6) $$\overline{\Delta\sigma'_v} = \tfrac{1}{2} \, \Delta\sigma'_{v_{max}}$$

Two sites where settlement is very evident have been considered to test the validity of the settlement calculation assumptions given above and have been labelled IGM BM 45 and 79. Accurate geotechnical investigations took place on the soil formation of these sites. The boring logs, the point resistance q_c of the cone penetration test (CPT) and some main geotechnical characteristics (unit weight of soil, Atterberg's limits, water content and RR) are reported in figures 6 and 7.

A cohesive formation, 3.5 m thick, overlays granular soil at the site IGM BM 45, while the cohesive formations reach depths of more than 10 m at the test site IGM BM 79. The plasticity index ($PI = w_L - w_p$) of the clayey soil ranges from 15 to 45 and RR ranges from 0.015 to 0.030.

The evaluation of settlement or heave, due to the maximum recorded, from 1976 to 1983, fluctuations of the free surface is carried out in the first 4 m of depth, since for this depth, differential measurements of levels (i.e., rod type bench mark level minus surface bench mark level) exist; the symbol for this difference of level is s (IGM-CGI BM). Piezometers, giving the pore water pressure, were also inserted at -4 m below the g.l. at each site.

The main data utilized to evaluate the difference of level in the considered depth are summarized in Table I.

TABLE I

IGM-CGI BM	Max. Fluctuation of Water Surface (m)	γ (t/m^3)	RR (x 10^{-2})	$\Delta\sigma'_v$ (t/m^2)
45	1.40	2.05	1.8-2.0	1.30
79	1.61	1.85-2.0	2.3-2.4	1.50

The comparison of computed and measured data (minima and maxima geodetic and piezometric values obtained during the six-year investigation were considered) is reported in Table II.

TABLE II

IGM-CGI BM	s computed (cm)	s(IGM-CGI) measured (cm)	s/s (IGM-CGI)
45	0.70	0.49	1.43
79	1.25	1.14	1.10

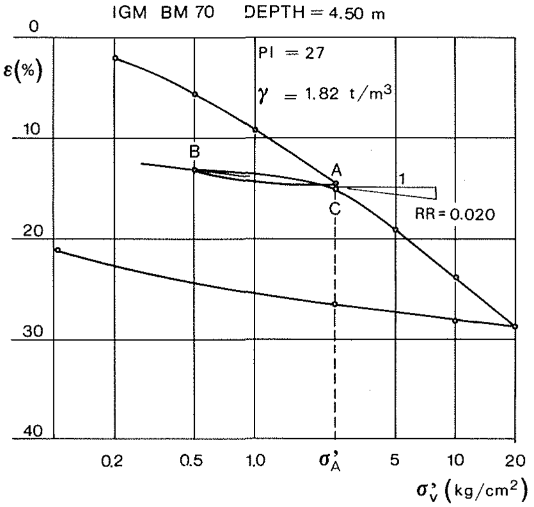

Fig. 4

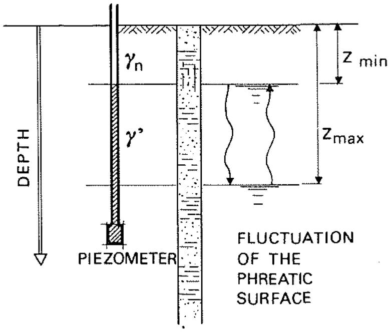

Fig. 5

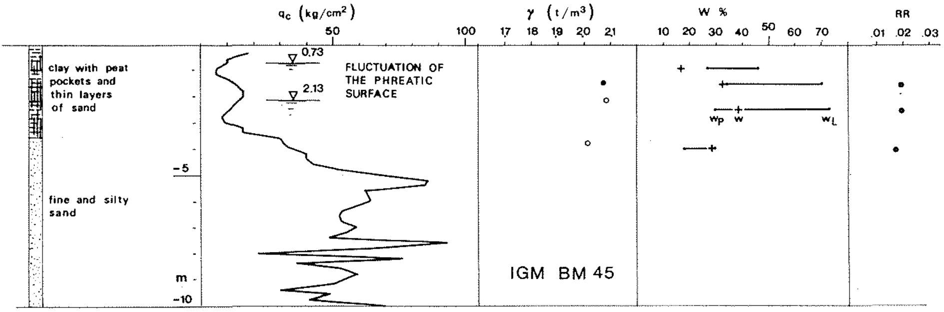

Fig. 6

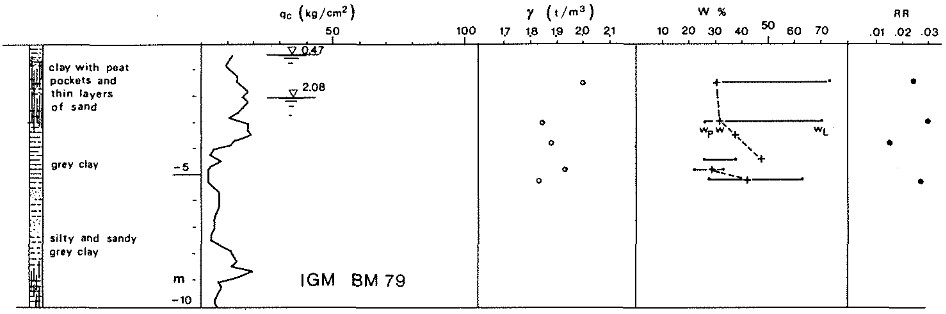

Fig. 7

The analysis of data shows that the computed and measured difference of levels are in close agreement.

The small discrepancies between 10-43 percent may be linked to variations caused by:

- sample disturbance both in the laboratory and on the site;
- time of action of the stress on the site, this could be too short to allow a complete consolidation of the cohesive soils; and
- time lag in the readings of the pore water pressures due to the type of piezometric point utilized.

Anyway, we must point out, that the measured level difference is only a part of the total settlement (or heave) of the bench mark on the land surface; as a matter of fact the tensional variation $\Delta\sigma'_v$ also effects deeper layers, other than the ones considered (i.e., down to 4 m), even if in an attenuated form and progressively decreasing with depth.

In soil mechanics a settlement analysis is usually carried out down to a depth which verifies the following equation

$$(7) \qquad \Delta\sigma'_v = 0.15 \ \sigma'_v$$

Through equation (4) we get the depth to be investigated

$$(8) \qquad H \simeq 6.7 \ \Delta\sigma'_v / \gamma'$$

Therefore for compressible soil formations such as the ones considered, fluctuations of the free surface of 1.5 m (as measured in the investigated sites) require anchorage of rod type bench marks to be at least 10 m below the ground level in order to achieve "stable" bench marks.

Data Analysis of the Other Bench Marks

The outcome of data from the study of bench marks 45 and 79 has encouraged us to extend the analyses to other bench marks kept under control.

Leaving out all the cases without hydrological findings and those with absolute movements so strong as to mask phenomenon under study, the attention was turned to 32 pairs of bench marks, of which 10 are placed in predominately sandy soils, 9 in clay or reclaimed land and 13 in mixed soil.

First of all the effect consequent to relative small settling movements or lifting due to factors other than hydrological was eliminated by computing the deviation of each value from the interpolated straight line which estimates the movement itself. Successively elevation differences and water table fluctuations measured parallelly during the 6-year study were compared for each pair of bench marks. After plotting the hydrological values on the x-coordinate and geodetic values on the y-coordinate, a straight line was interpolated according to least squares and confidence limits were estimated. It resulted that all the investigated bench marks underwent the movements moving in the same direction of piezometric variations, of a magnitude varying with the nature of the soil. Analyzing in fact these graphs one notes that the straight lines representative of the behavior of bench marks placed in analogous soils (by weighted averages) grouped together in sheaves with a minimum slope for sand and a maximum for clay. Figure 8 shows the mean of straight lines reconstructed for three groups of bench marks anchored into three types of soils considered. Their slope assumes the value of $-0.92 \times 10^{-4} \pm 0.30 \times 10^{-4}$ for sand (usual standard deviation), of $-18.88 \times 10^{-4} \pm 0.90 \times 10^{-4}$ for clay and $-4.84 \times 10^{-4} \pm 0.42 \times 10^{-4}$ for alternation of the two soil types. Thus it seems characteristic for each type of soil considered, depending on its different mechanical behavior and to changes in the water table, to cuase definite surface movements.

If we compare the straight line representative of clay with the ones of bench marks 45 and 79 (Fig. 3), it is quite clear that the latter have the greatest slope, representing the behavior of the more compressible clays found.

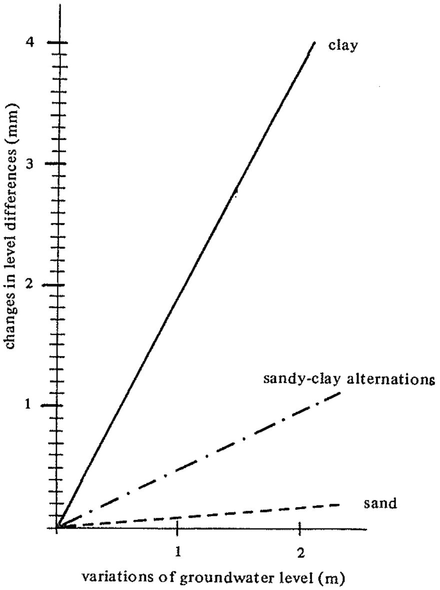

Fig. 8. Trend of correlation between elevation differences of the benchmarks and water table fluctuations in the soils considered.

Conclusions

The investigation defines the behavior of bench marks superficially placed in sedimentary soils associated with fluctuations of the water table. It showed that these bench marks were subjected to periodic movements of lifting during rainy spells and settling during dry ones. The magnitude of these movements vary with the lithological nature of the soil into which the bench marks were established and it is minimum for sand with a mean vertical movement less than 0.1 mm for 1 m of piezometric variation and maximum for the same piezometric variation for clay with the mean vertical movement of 2 mm and maximum of 10 mm, in regards to the different compressibility of these soils. The bench marks located in sand and clay alternations obviously have vertical movements between the two extremes indicated.

The results from the investigation could bring about the suitable method in establishing a new leveling net, used as precise criteria in carrying out the levelings along the already existing survey network. Since stability of bench marks established into incoherent soils is due to other causes (which form the objective of a broader study for which a lot of data have already been collected), it seems appropriate to wait for the results from processing in order to reach a comprehensive conclusion which will involve the entire problem in all its aspects.

Acknowledgments

The authors wish to thank Alberto Tomasin of ISDGM-CNR, Venice, for supplying the tidal data.

RESULTS OF RECENT LEVELLINGS IN THE REGION OF MODENA

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Abstract

The analysis of levellings carried out in the period 1950-1980 shows that a remarkable soil sinking hase taken place in the region of Modena. As data concern only a levelling line, the Municipality of Modena has set up a levelling network in 1981 in order to study the phenomenon. The layout of such network is described and the results of the first survey are discussed, with particular regard to accuracy attained in measurements. Finally new values about subsidence resulting from the first survey and from a partial survey carried out in 1982 are presented.

Introduction

In 1979 remarkable sinkings of some levelling benchmarks located in Modena and in its suburbs, were discovered. This was pointed out by the comparison between the results of two levellings performed in 1950 and 1974 by I.G.M.I. (Istituto Geografico Militare Italiano) and Cadastre along the line n.17 Bologna-Parma of the national levelling network.

Though it had not been measured the vertical movement of soil had already revealed itself. Several damages in fact had been noticed on hystorical buildings of the city for some time.

A research of all available levelling data regarding Modena area was carried out to get as many information as possible on subsidence. Unfortunately it resulted that the only line n.17 had been relevelled some times.

The graph of the variation of heights of the benchmarks of line n. 17 detected in various campaigns of measurements starting from 1950 is plotted on fig.1.

Although the heights of control points should be referred to a common origin, however stable linkage bench marks have not been found. An investigation about possible movements of various reference bench marks has suggested us to ascribe an indetermination of a few centimeters to data of levellings performed in 1972,1973,1979 and 1980.

Fig. 1 shows that the greatest sinkings have occurred in an area which extends for about 12 km from east to west; the maximum subsidence value is cm 81.8 from 1950 to 1979.

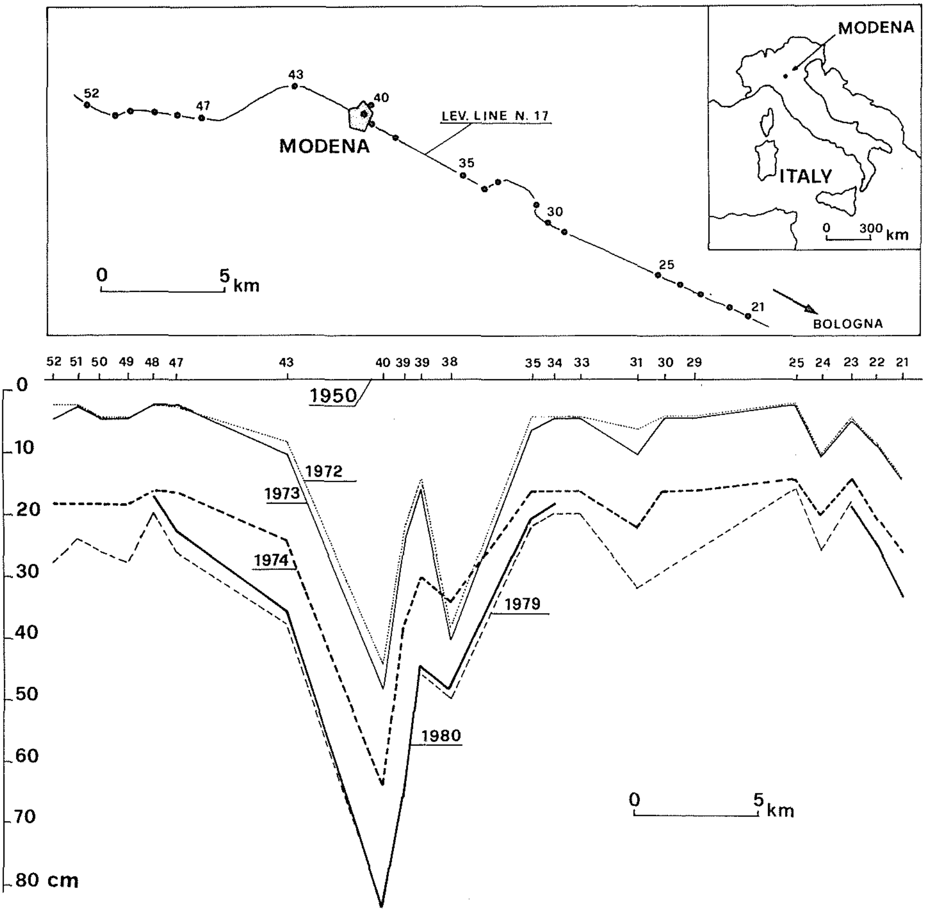

Fig.1 Displacement profiles concerning levelling line n. 17 Bologna-Parma of I.G.M.I.

With regard to the rate of soil sinking it is advisable to consider a long period as we do not know with accuracy the soil movements occurred between the two more recent sets of measurement (1979,1980) because of the different reference bench marks assumed.

The maximum sinking velocity has resulted about 4 cm/year between 1972 and 1980, in the linear hypothesis, as it showed by the diagram time/subsidence concerning bench mark n. 17/40 (fig.2).

The new levelling network of Modena plane

In consideration of such remarkable soil vertical movements and of their possible consequences, the municipality of Modena

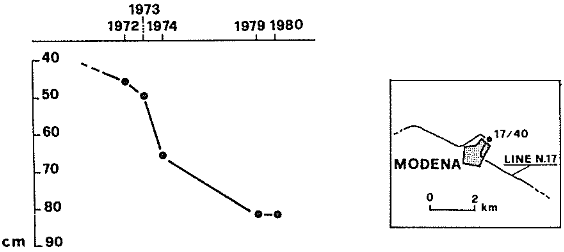

Fig.2 Diagram time/subsidence of the bench mark n. 17/40.

in 1981 decided to set up a levelling control network, with the following aims: detecting soil movements in detail in urban area and checking the extent of subsidence in the surrounding plane.

The territory which is object of study has a surface of about 350 km^2 and includes the area in which the greatest soil sinking occurred and the most important industrial centres are located.

The course of levelling lines has been planned in order to comprise several old benchmarks whose heights are known at certain epochs. In this way new data on soil movements can be obtained since from the first survey. The new benchmarks have been placed in steady buildings on condition that they are not recent or susceptible of modification and far from rivers or reclaimed lands .

The levelling network itself is made by the city area network (fig.3) and by the esternal area network (fig.4) .

In the former the average length of sections is 0.25 km, while the length of loops is about 2.5 km, in the latter such lengths are respectively 1 km and 30 km on average.The overall length of levelling lines is 160 km and 225 is the number of new benchmarks.

In the plane surrounding Modena soil is sinking also at considerable distance from the town. Therefore the reference benchmark (P/20) of the network has been placed in the slopes of the mountains (Appennino Modenese) . The locality selected was considered stable, in relation to the subsidence of the plane, by geologists and it is nearer the area under control than other ones suitable to receive reference benchmarks. Moreover the levelling line which links the reference bench mark to the network runs for several kilometers over stable gravelly ground. A few control points were placed in emerging rocks near the reference bench mark.

In the first survey of the network, the levelling lines

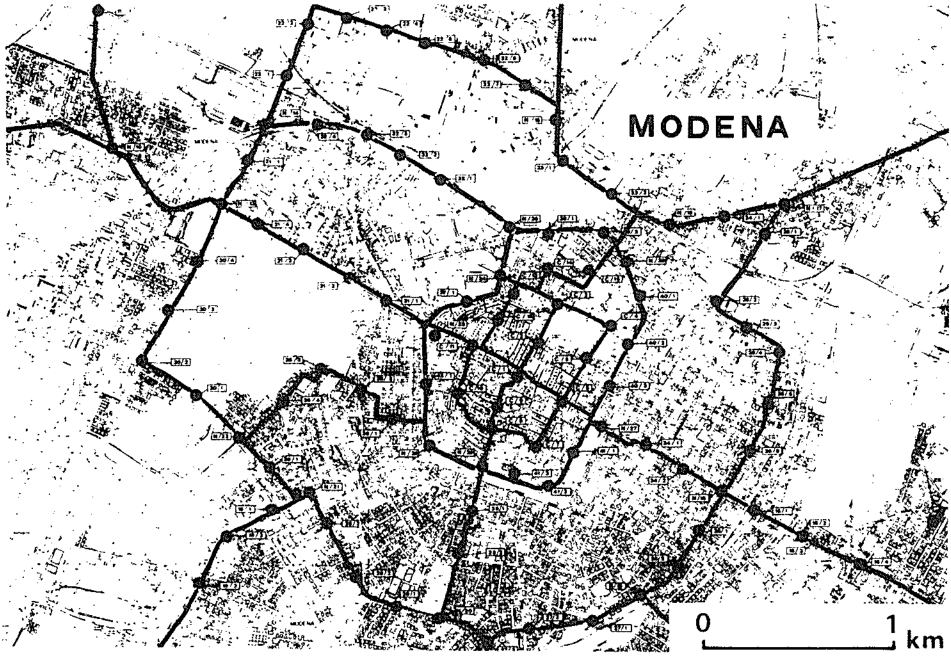

Fig. 3 Modena city levelling network

n.17 and n. 5 of I.G.M.I. have been levelled in order to *connect the bench mark n.P/20 to the bench mark n.5/163 (fig.5).* This point is situated in Appennines near Bologna city, it has been assumed as reference bench mark by Cadastre in 1974 and it is near the reference bench mark of I.G.M.I. levelling performed in 1980. By this way the heights of bench marks of Modena network can be used for any technical purpose being congruent with national levelling network heights.

Survey of the network

The first survey of Modena levelling network has been carried out in 1981. Equipment and procedures of special order levellings have been used. In particular the allowable discrepancy between indipendent forward and backward levellings between bench mark has been $\pm$ 3 mm $\sqrt{D}$, where D is the distance in km between bench marks measured along the levelling route.

Measurements have been performed as quickly as possible to reduce the effects of soil movement on the results of levelling. Observed differences of heights have been corrected of rod graduation errors, even if they proved to be very little . No correction due to atmospheric refraction has been made, because of the negligible slope of levelling routes and

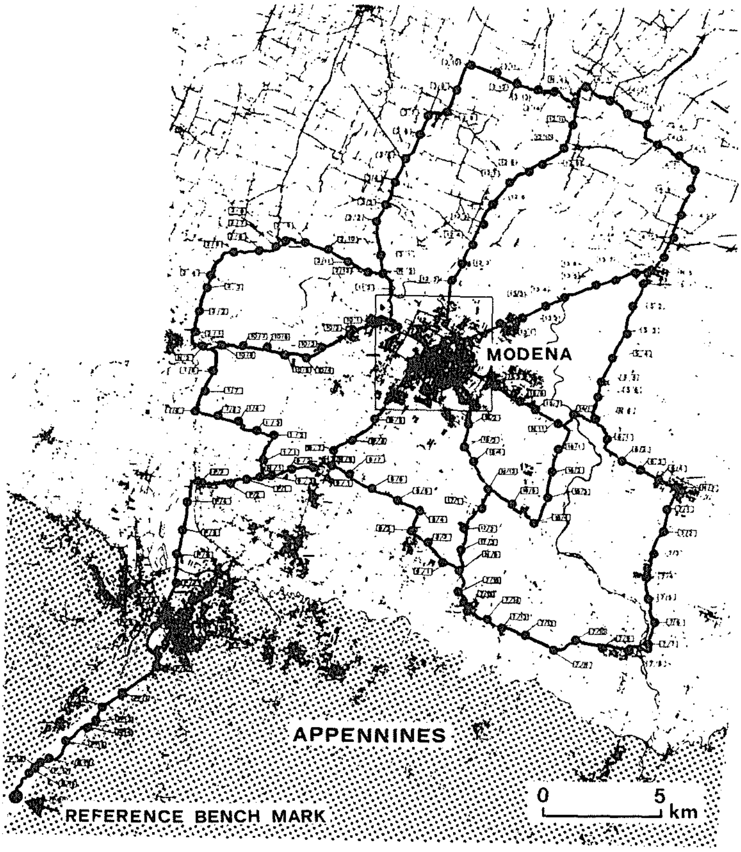

Fig.4 Modena plane levelling network.

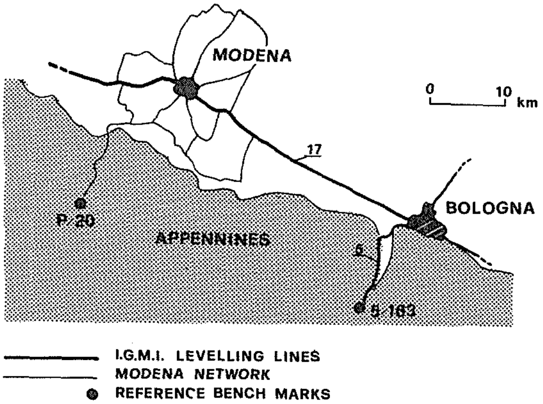

Fig.5 Scheme of the junction of the levelling networks of Modena and of I.G.M.I.

of the specified minimum sight ground clearence (0.5 m) and maximum sight lenght (40 m).

Some preliminary controls on accuracy attained in levelling have been kept before adjustment.

First of all the standard deviation S*(mm) of measured differences of heights along a unit distance (1 km) has been computed using the formula:

$$|1| \qquad S^* = \frac{1}{2} \sqrt{\frac{\sum_{i=1}^{n} d_i^2/l_i}{n}}$$

where: d_i(mm) is the difference between forward and backward levelling of the ith section, l_i(km) is its length and n is the number of sections. The values of S* obtained considering the network of the city area of the external area and the whole network separately are listed in tab.1.

TAB. 1

	S*(mm)	n
City area network	0.31	102
External area network	0.51	156
Whole network	0.46	258

It is interesting to note that S* is remarkably lower in the city where the average length of the connection lines between two adiacent bench marks is 0.25 km, than in the external area where such distance is 1 km on average.

The second control has been made to verify if levelling loops misclosures exeeded acceptable limits.For this purpose wariable $W/\sqrt{L}$,where W(mm) is the misclosure of a levelling loop and L(km) is its perimeter,have been considered.The values of W have been computed from levelled height differences without gravity conections because the influence of earth's gravity field on loop misclosures is negligible in the area of Modena, owing to its limited surface and to the slight slope of terrain. In order to assume the observed values of such variable distributed according to a normal distribution,we have used the following procedure.First of all the mean $\bar{x}$ and the standard deviation S_o have been calculated from data,obtaining $\bar{x} = 0.15$ mm and $S_o = 0.94$ mm;then the interval $(-\infty,+\infty)$,has been divided into 4 class intervals,for each one of them the theoretical probability is p = 0.25.This means that beeing 22 the size of the sample, the theoretical frequency for each interval is f_i=5.5.Finally the fitted frequencies f_i have been compared with the corresponding observed frequencies f_{oi} using the two-sided χ^2 test.As 1 is the number of the degrees of freedom,the value of $x^2 = \sum_{i=1}^{4} (f_{oi} - f_i)^2 \cdot f_i^{-1}$ is 1.64. Stating a significance level $\alpha = 0.05$ is $\chi^2_{1.0025} = 0.001$ and $\chi^2_{1,0.975} = 5.02$ and this means that the value of x^2 is evidently not significantly large.

Then assuming the normal distribution for $W/\sqrt{L}$ it can be reasonably fixed the confidence interval $(\bar{x} - 2S_o\sqrt{L}, \bar{x} + 2S_o\sqrt{L})$, i.e. $(0.15 - 1.88\sqrt{L}, 0.15 + 1.88\sqrt{L})$mm, for loop misclosures W. Only two values of W do not lie within such interval, although they are comprised in the interval $(\bar{x} - 3S_o\sqrt{L}, \bar{x} + 3S_o\sqrt{L})$. In conclusion the hypothesis of the presence of blunders in measurements has been rejected, and all levelling data have been accepted for adjustment.

Finally it is interesting to make the following remark about the presence of systematic effects.Under the assumption of statistical independence of measured height differences ΔH between the end points of individual segments,it is known that variance of ΔH can be expressed by $\sigma^2_{\Delta H} = \sigma^2_1\ l$,where σ^2_1 is the variance evaluated for a distance of 1 km and l is the length of the segment.This means that the expected variance σ^2_W of W is $\sigma^2_W = \sigma^2_1 L$ and if W's have a normal distribution N $(0, \sigma^2_W)$ then the stadarized circuit misclosures $\bar{W} = W/\sigma_1\sqrt{L}$ have a standard normal distribution N(0,1). Now the standard deviation $\bar{S}$ of the values of $W/\sigma_1\sqrt{L}$, computed after having assumed $\sigma_1 = 0.46$ mm(see tab.1), results $\bar{S} = 2.14$ mm. It is easy to proove that this value is significantly larger then 1. This fact is considered as a demonstration that the basic assumption of statistical independence of individual height differences between consecutive bench marks along a line is not satisfied (Vaníček, Krakywsky, 1982). This also explains the difference between the standard deviation S* evaluated by |1| and S_o computed from misclosures.

The outlined analysis points out that levelling data can be assumed in the adjustment of the network, even if they are affected by some systematic errors. By consequence the application to Modena network of adjustment methods (Vanicek, Krakiwsky,1982) in which observed differences of heights are considered as correlated quantities could give interesting results. For now measurements have been adjusted by least squares method under the statystical independence hypothesis. This is also justified by the probable slight exetent of systematic errors because of the little values of standard deviation of observed height differences estimated using discrepancies and misclosures. Moreover we have got new data on soil vertical movement in a short time.

The standard deviations for a distance of 1 km which results from adjustment is s = 0.943 mm while the standard deviations of the heights of the bench marks have been computed with regard to two different points: the bench mark P/20 (S.

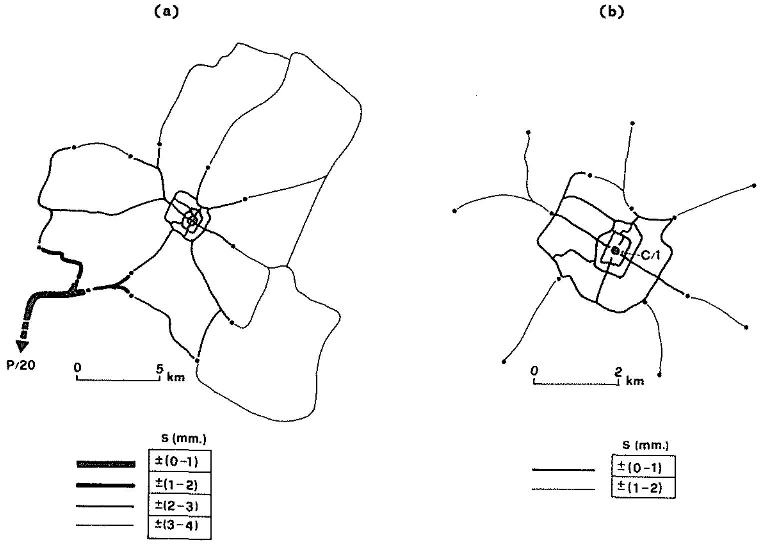

Fig.6 Standard deviation **s** of heights resulting from adjustment; (a) reference bench mark: P/20, (b) reference bench mark: C/1.

Michele M.) which will be assumed as reference bench mark in next relevellings of the network, and the bench mark C/1 (Modena, Civic Tower) which is located just in the center of the city area. This procedure allows us to evaluate the significance level of soil sinkings in the whole territory and of relative soil vertical movements in the urban area, where damages of buildings produced by differential movements of foundations are frequent. Fig.6 shows that least squares estimated standard deviations does not exceed 4 mm, and 1 mm in the first (a) and in the second (b) case respectively.

Recent soil vertical movements (1981-1982)

The municipality of Modena set up a levelling network in 1962 for technical purposes. Several bench marks of that network have been relevelled in 1981. Therefore it has been possible to determine contour lines of equal subsidence for the period 1962-1981 (fig.7). The soil sinkings rappresented by such lines may be affected by a systematic error, whose value should not exceed a few centimeters, due to possible little movements of the reference bench marks assumed in the levelling carried out in 1962. In any case the graph of fig.7 indicates that the more remarkable vertical movements (55-85 cm) took place in the northern part of the town and they are distributed in an area which stretches from east to west for

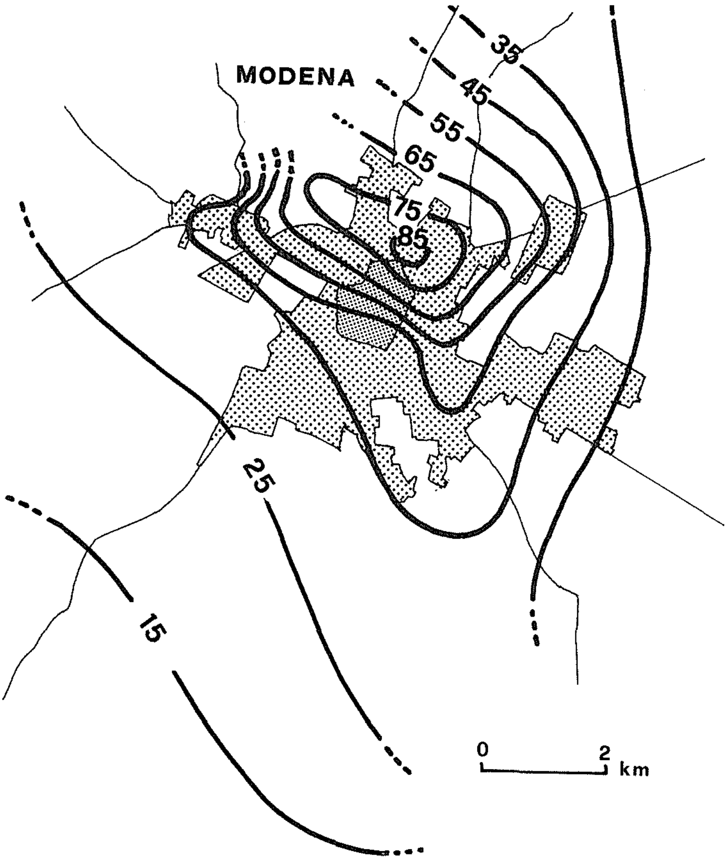

Fig. 7 Contour lines (in cm) of equal subsidence in the period 1962-1981.

4 km at least and has a surface not inferior then 12 km^2.

New data about subsidence are also given by the displacement profile along levelling line n.17 Bologna-Parma (fig.8). The levelling performed by Cadastre in 1974 has been assumed as reference in order to use a greater number of bench marks to describe the phenomenon and to point out only recent movements. Fig.8 shows that subsidence involves the whole urban area of Modena and in particular the hystorical centre where soil sinking comes to 17.9 cm: soil appears stable out of the town, in north-west and south-east areas. In the same figure the profiles resulting from 1979 and 1980 levellings are also plotted. Although results of the recent surveys are congruent however a significant variation of movements from 1979 to 1981 does not appear , while we have seen that phenomenon rate is about 2-2.5 cm/year in the city area.

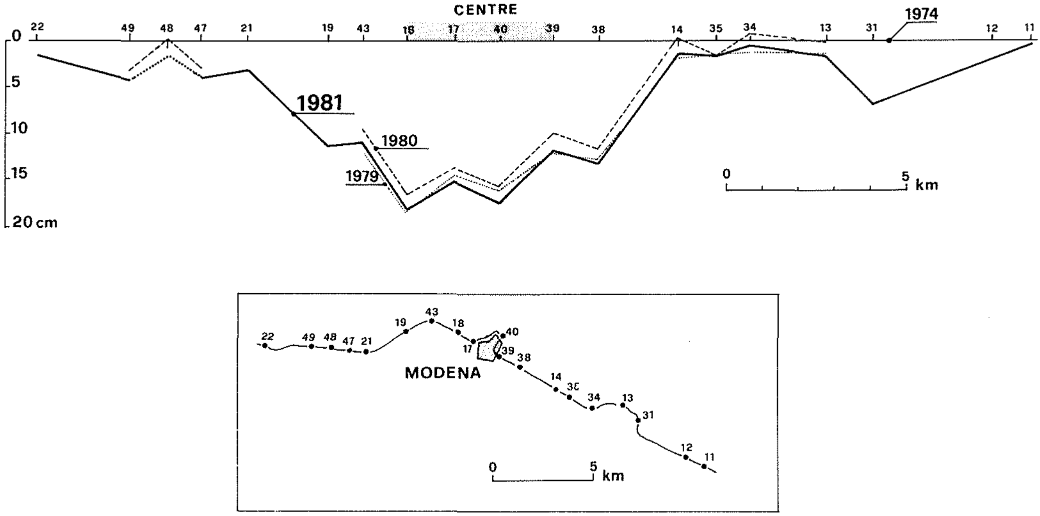

Fig.8 Displacement profiles along levelling line n. 17 in the period 1974-1981.

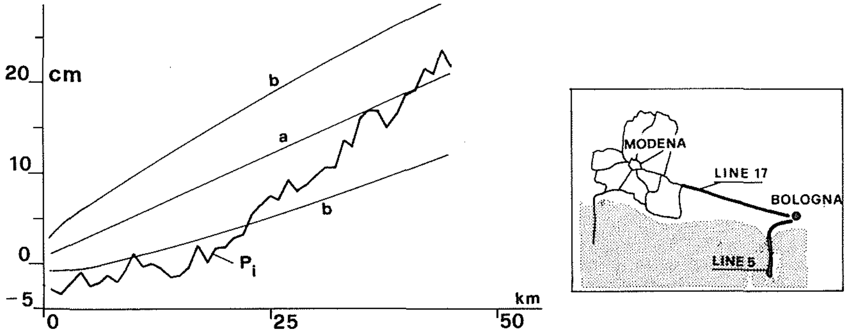

Fig.9 Diagram of P_i,accumulated discrepancies,concerning levelling line n.5 and n.17; line (a) represents the computed mean of P_i and line (b) its standard deviation.

This can be explained if we consider that heights of fig.8 are referred to the stable bench mark n.5/163 which is about 60 km far from Modena and which is linked to the levelling network by a branch line. By consequence the influence of possible unmodelled systematic errors,above all, may be noticeable enough. To prove this the graph of $P_i = \sum_{i=1}^{n} \varrho_i$ is reported in fig.9, where ϱ_i is the discrepancy of the i^{th} section and i is the number of adiacent sections starting from the first one of the levelling lines n.5 and n. 17 of I.G.M.I.

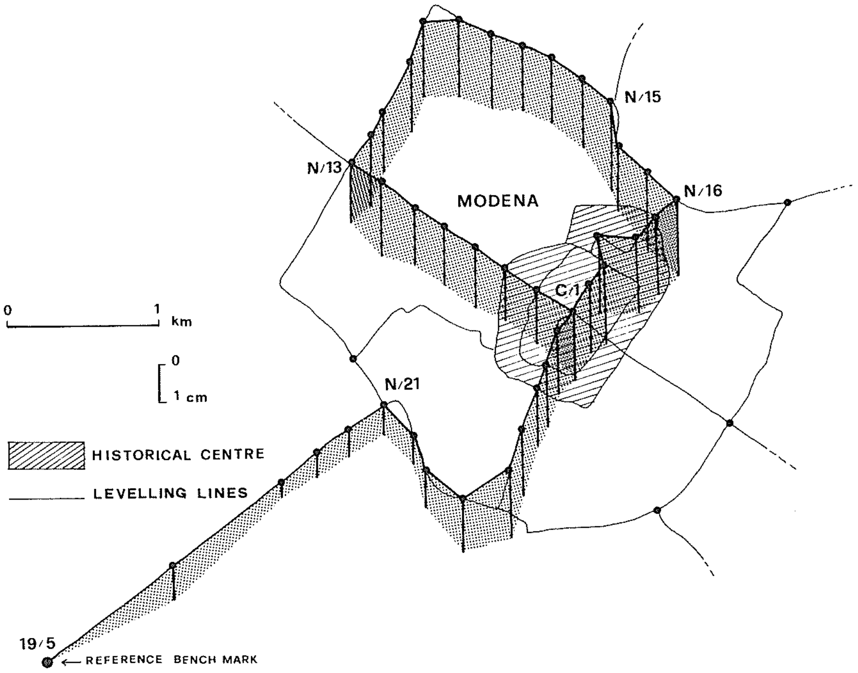

Fig.10 Variation of heights of some bench marks of Modena Network in the period 1981-1982.

indicated in the same figure. The average slant of the line a(estimated mean of P_i) indicates the presence of systematic effects in measurements (Pieri,Chiarini 1967).

This problem fortunately regards only old data because the new reference bench mark which will be used in next relevelling is close to Modena network.

Anyhow waiting for a new survey of the whole network, some levelling lines located in the city area have been relevelled in 1982 in order to get some recent data about relative soil vertical movements. Measurements have been carried out under the same technical specifications used in 1981.
Variation of heights reported in fig.10 are referred to the bench mark 19/5 which is supposed to be the most stable within the surveyed area. Movements appear to increase northwards; the maximum value is 2.5 cm (bench mark n.N/15).

Conclusions

The survey of Modena levelling network outlined in the present communication is a zero-measurement. Therefore we have to wait for a relevelling to know soil movements.

For now it can be pointed out that a good accuracy has been

attained in the first levelling, even if the analysis of data has showed the presence of some systematic effects in measurements.
Then it will be necessary to reduce the influence of such errors in next field operations and to consider them in adjustment of the network.

With regard to subsidence, the results of new levellings performed in 1981 and 1982 give us an idea of areal distribution of soil sinkings and indicate that their actual values are not less then 2.5 cm/year in the city area.

Acknowledgements

I have to thank the municipality of Modena, Dr. M. Barbarella of the Istituto di Topografia, Geodesia e Geofisica Mineraria of Bologna University who elaborated the computer program for the adjustment of the network and the Land Surveyor P. Barbieri , who gave me a great deal of help in this study.

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ACOUSTIC EMISSION MONITORING OF LAND SUBSIDENCE

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Abstract

Acoustic emissions are sounds generated internally in a soil or rock mass as it deforms. These sounds produce stress waves which stimulate transducers implanted at strategic points in, or on, the material being monitored. The paper presents four case histories on relatively local subsidence which utilize the technique. For situations where larger land masses are involved, i.e., areal subsidence, the sensors must be left in place over long time periods and periodically visited. In such cases remote sensing is a definite possibility. Six schemes are presented, based on either continuous or periodic transmission, which include direct coupling, telephone and airborne concepts.

Problem Definition and Scope

While ground subsidence caused by subsurface disturbances has long been recognized as being devastating to structures, the subject has only recently been approached on a unified and concentrated basis, e.g., refer to various Conference Proceedings in 1969, 1976 and 1978. As shown in Table 1, the area can be categorized via the various mechanisms involved along with their range of dimensions and approximate mobilization times for such mechanisms to occur.

Table 1 - Subsidence Mechanisms, Typical Dimensions and Time For Mobilization, after Scott, 1978.

Mechanism	Dimension, km	Time, years
Crustal isostatic adjustments	1000-100	10,000
Plate tectonics, volcanic	. . .	100-10
Large-scale water withdrawal	. . .	100-10
Earthquake	. . .	10^{-6}-10^{-7}
Fluid withdrawal	100-10	100-10
Liquid or solids extraction	10-1	100-10
Surface loading, consolidation, hydrocompaction	1-0.1	100-10
Vibration densification	1-0.1	10^{-6}-10^{-7}
Compaction	0.1-0.01	0.1

In all of these situations it is necessary to postulate the nature of the occurrences vis-a-vis the potential impact on the structure(s) in question. While difficult to accomplish, such work is ongoing as per the references cited. Needed, however, are proper monitoring techniques to provide input data and parameters into such models. Additionally, in the absence of suitable subsidence models, these field monitored values become the model itself, i.e., the development of an empirical model. In its simplicity, this technique could result in a graph of vertical ground deformation versus time and when a distinct break in the curve is noted, or an intolerable settlement reached, remedial action could begin. The problem with this type of approach is that the typical response is usually piecewise linear with no obvious break occurring before major subsidence results in intolerable situations.

Needed is a technique which affords a precursor, or early warning, of rapdily increasing subsidence. For instance, if a field monitored parameter responded as a three-piece linear plot versus time, the entry into the second stage would be a forewarning of failure. Certainly whatever the technique it should be more sensitive than the usual methods of measuring linear deformation.

It is the opinion of the author that such a technique is available and is based on the subaudible sounds generated by ground instability. Monitoring of these sounds, called acoustic emissions, is the focus of this paper. The paper will describe the method along with a number of situations involving subsidence. A series of remote sensing schemes will then be offered along with a summary and conclusions.

Acoustic Emission Overview

Acoustic emissions (AE) are sounds generated within a material which has been stressed and subsequently deforms. Sometimes these sounds are audible (wood cracking, ice expanding, soil and rock particles abrading against one another, etc.), but more often they are not, due to their low amplitude or high frequency, or both. A piezoelectric sensor is used as a "pickup" to detect the emissions. This transducer produces an electrical signal proportional to the amplitude of sound or vibration being detected. The signal is then amplified, filtered, and counted or recorded in some quantifiable manner. Unwanted machine and environmental noise is electronically filtered from the signal or separately quantified and subtracted from the test results. The counts, or recordings, of the emissions are then correlated with the basic material being tested. Usually, if no emissions are present, the material is in equilibrium and thus stable. If, however, emissions are observed, a nonequilibrium situation exists which may eventually lead to failure of the material.

It should be mentioned at the outset that the technique is not new. Only its application to soil masses and related problems is new. The original soils references being in the late 1960's, and our work commencing in 1971. See Koerner, et al. (1981a) for a summary of this work. Of particular interest to those considering use of the technique on rock problems is the work done in the mining industry. The leading organization appears to be the U. S. Bureau of Mines, but excellent work has been reported in Canada, West Germany, Poland, Russia, Australia, and Japan. Many of these projects have been described in the Proceedings of three Conferences held at the Pennsylvania State University in 1975, 1978 and 1981.

Equipment Details

The components of an AE monitoring system consist of a wave guide (to bring the signals from within the soil to a convenient monitoring point); sensor (geophone, accelerometer, hydrophone or transducer to convert the mechanical wave to an electrical signal); preamplifier (to amplify the signal if a long cable is being used); filters (to eliminate undesirable portions of the signal); amplifier (to further amplify the signal for processing); and a quantification system. The quantification system usually is some form of counter and can be accomplished in one of the following ways.

- ringdown counts (or count rate)
- event counts (or event rate)
- amplitude (or amplitude distribution)

- energy (RMS or other type of analysis)
- rise time
- frequency analysis

The variations of equipment configurations can obviously be great which leads to many possible systems. Currently a number of companies are making single channel AE systems for geotechnical use (Acoustic Emission Technology Corp., Dunegan/Endevco, W. Nold Co., Slope Indicator Co., Weston Geophysical Corp.) and some are making multi-channel systems (Acoustic Emission Technology Corp., Physical Acoustics Corp., and Slope Indicator Co.). The option also exists of making a hybrid system consisting of commercially available components from any one of a number of equipment manufacturers.

AE in Subsidence Related Problems

In this section, four different situations will be presented where AE has been used in monitoring subsidence related problems. In the first two, deformations were monitored along with AE so that direct comparisons of the two methods can be gained. AE instrumentation in each situation differed somewhat, so that each case history should be viewed independently.

(a) Laboratory Modeling of Footing Deformations (Lord, et al., 1976)

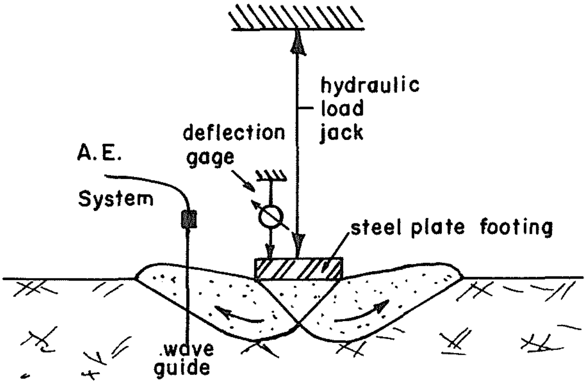
One can gain a considerable amount of information from laboratory model tests showing how a soil bearing capacity failure occurs beneath shallow foundations. In such an test, soil is placed in a narrow tank in marked layers under carefully controlled conditions. A steel plate, simulating a building footing, is placed on the soil as shown in Fig. 1(a). Deflection gages are mounted on the footing and a hydraulic jack then presses the footing into the soil beneath it. Such load is continued until a failure occurs (load begins to drop off) as is clearly seen by the mobilized slip line pattern.

We have added to this standard bearing capacity test by also monitoring the AE generated during the process. The wave guide (12 in. long, 1/8 in. diameter steel rod) was placed into the soil intersecting the ultimate shear plane as seen in Fig. 1(a). Load, deflection and AE counts were taken, resulting in the curves of Fig. 1(b). There the following observations can be made.

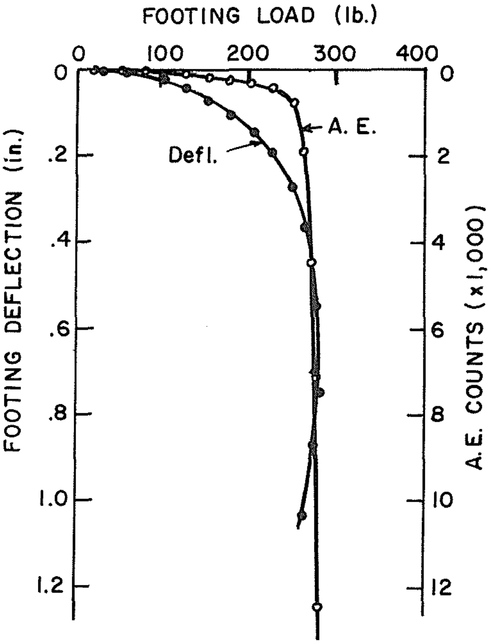
- Both load vs deflection and load vs AE curves show the same general type of behavior.
- The load vs AE curves is more abrupt in its entering into the plastic state. This seems to indicate that AE monitoring is more sensitive to soil deformation than is surface deflection monitoring, and gives a more clearly defined value of the ultimate load.
- The test was reproduced a number of times with almost identical results.

(b) Consolidation Settlement Due to Surcharge Fill (Koerner, et al., 1978)

This case history represented a field study at the Philadelphia International Airport in Philadelphia, Pennsylvania and is shown in schematic form in Fig. 2(a). A surcharge fill was placed around a previously installed end bearing pile to determine how much load would be added to the pile due to soil consolidation. This test constitutes a full scale negative skin friction, or downdrag, test and represented an opportunity to monitor AE generation during deformation of the clayey silt undergoing consolidation. The test piles and settlement anchors were employed as AE wave guides. A separate exterior casing was used to prevent contact between the soil and the anchor except at its tip. Fig. 2(b) presents the test results, where

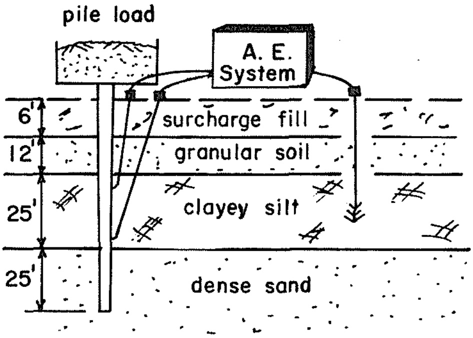


(a) General Schematic of Test Setup

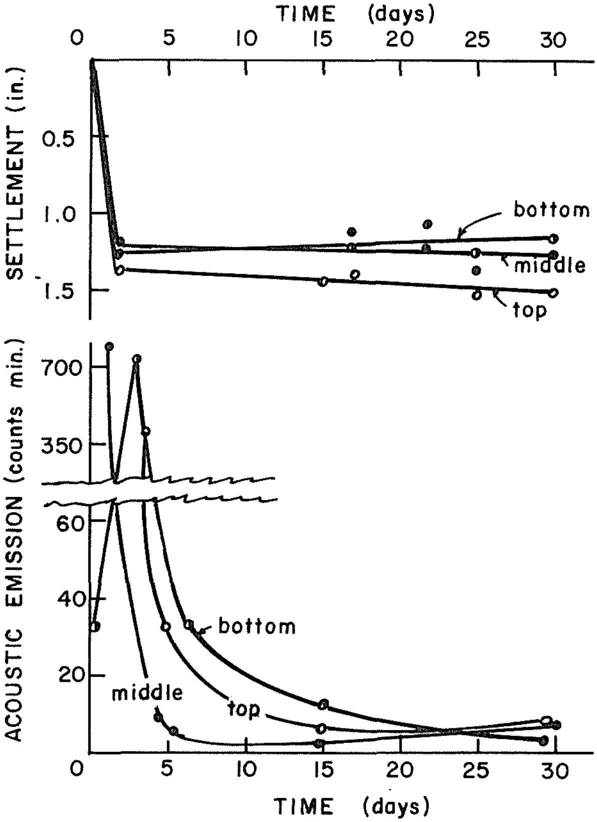


(b) Acoustic Emission and Deformation Response Curves

Fig. 1. - Laboratory Modeling of Footing Deformations



(a) General Schematic of Test Setup



(b) Acoustic Emission and Settlement Response Curves

Fig. 2. - Consolidation Settlement Due to Surcharge Fill in Philadelphia, Pennsylvania

the general similarity between the settlement vs time and AE vs time response curves should be noted. The fact that the AE response dissipated after 5 days-15 days is actually in better agreement with theoretical computations, using standard consolidation theory, than the 2 days- 3 days for the measured settlement response. The reasons for the midlayer response reaching equilibrium in a shorter time than the upper or lower portion of the stratum is not known.

The most significant feature of the test is that the soil was very emittive while in an active state of deformation. Conversely, when the soil came to equilibrium the acoustic emissions essentially ceased.

(c) Foundation Soil Instability (Koerner, et al., 1981b)
This site consists of a system of holding ponds for various chemical waste liquids. The embankments vary in height from 8 ft. - 20 ft., have steep side slopes (about 1 on 1), and are founded on extremely poor foundation soil. These foundation soils are silty clays and clayey silts with standard penetration resistances from 0 to 5 blows/ft. A deep seated base stability failure had occurred at the site before AE monitoring began. The acidic wastes being impounded (about 2,000,000 gal) emptied into the river that flows adjacent to the site. Upon repair of the failed dike, the ponds were again filled with similar liquids. The site has since been monitored using 12, 1/2-in. diameter wave guides that were easily installed by pushing 4-ft. sections into the foundation soils to depths up to 20 ft. Subsequent monitoring has shown that AE activity is usually present, but that count rates vary considerably. As an example, the center wave guide at the toe of the slope has resulted in the following AE information during periods of intermittent monitoring.

Table 2 - AE Count Rates of Fine Grained Foundation
Soils Beneath Small Earth Dam in New Jersey

Date	AE Count Rate
July, 1975	Failure and Reconstruction of Dam
September, 1975	10 to 30 AE counts/minute
October, 1975	0 to 7 AE counts/minute
November, 1975	0 to 7 AE counts/minute
February, 1976	20 to 40 AE counts/minute
March, 1976	5 to 10 AE counts/minute
June, 1976	10 to 18 AE counts/minute
September, 1976	5 to 10 AE counts/minute
October, 1977	5 to 20 AE counts/minute
March, 1978	4 to 15 AE counts/minute
May, 1978	10 to 20 AE counts/minute

With this information, the owner was encouraged to monitor the site continuously with a triggered alarm on the AE System set at 50 counts per minute. This sytem is currently operative with an engineer visually inspecting the site whenever the alarm is sounded.

(d) Coal Mine Subsidence Monitoring Beneath Building (Lord, et al., 1976)
Regarding building foundation monitoring one structure has been proposed for evaluation and preliminary work has been performed. This particular structure is a five-story apartment building founded over a series of worked-out coal mines. The entire area has a history of subsidence problems where abrupt settlements of up to 15 ft. at the ground surface have occurred. Two sets of 30 ft. long, 1/2 in. diameter steel rod wave guides were installed on a temporary basis. AE rates of 2 to 100 counts/minute

were recorded. In view of this relatively active situation a more elaborate and permanent scheme as shown in Fig. 3 has been proposed to the owner. Both the concrete structure and the soil/rock system beneath the footings are intended for monitoring. Continuous recording type monitoring with eight channels will be used for this particular site. It is anticipated that AE rates would substantially increase if deformation of the subsurface began to occur. Conversely, if little or no emissions were recorded the situation would probably be stable.

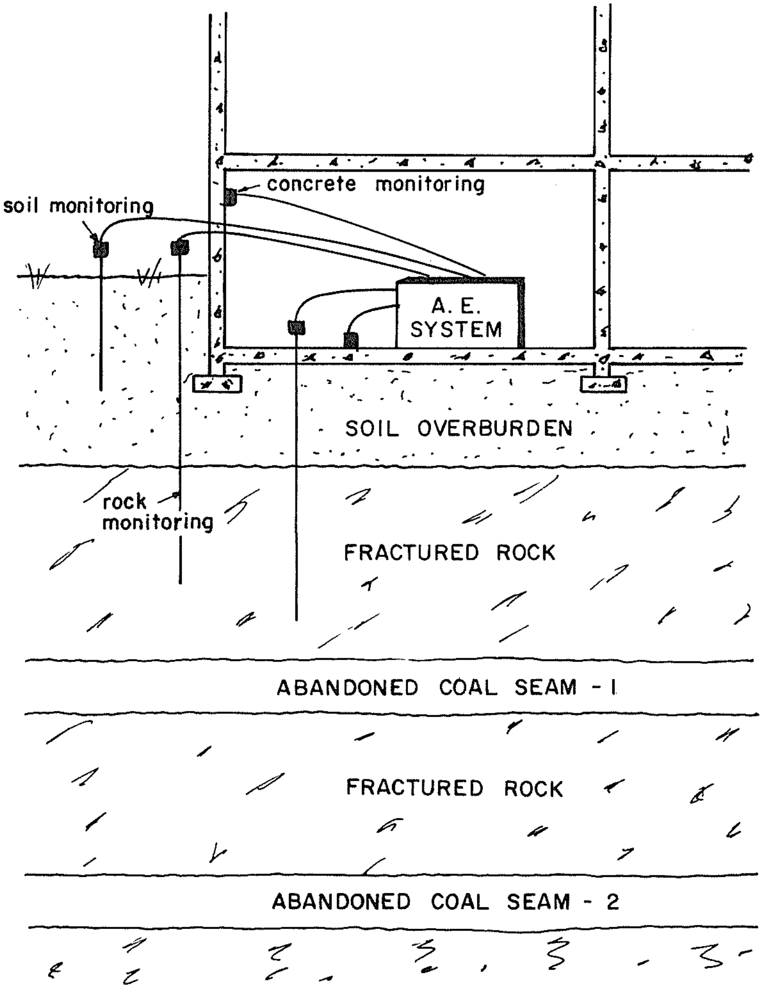


Fig. 3. - Proposed Acoustic Emission Monitoring of Soil and Rock Foundation Beneath a Building Along with Concrete Monitoring of the Structure in an Abandoned Mine Subsidence Area

Areal Subsidence and Remote Sensing

For situations where large land areas are involved in the subsidence action, or in remote areas where accessibility is a problem, AE monitoring will be somewhat different than just described. In such cases the wave guide and sensor assembly is the same, but now decisions on AE signal transmission must be made. Two different possibilities exist:

- Continuous Transmission - In this situation the incoming AE data is continuously transmitted to a receiving station as it occurs. Interpretation can be made as data is received.
- Periodic Transmission - In this situation the incoming AE data is stored and identified at the site where it is generated and then periodically transmitted. This transmission can be made to a ground station, or to an overflying plane or satellite.

Obviously, there are many possible variations of the above schemes. In addition to the transmission details, the problem of supplying on-site power is of concern. In this case of continuous transmission, power is needed for the AE system, timer and transmitter. For periodic transmission, power is needed for the above items and for a data processing and storage system. Some thought has been given to generalized configurations, realizing that each case will be site-specific to varying degrees.

For a remote monitoring system, left unattended for long periods of time (months to years), a number of possibilities are envisioned under the following assumptions.

- Remote site power is provided by a photovoltaic (solar) cell array and backup batteries would be charged by these calls.
- All monitoring units would be subjected to ambient, hence harsh, weather and environmental conditions.
- Multiple pickup sensors will be emphasized.
- Base station would have minimum human attention.

Using the above constraints, a number of possible schemes can be discussed. In each case, AE signal transmission becomes more complex with the important advantage, however, that less data interpretation is required.

1. Continuous transmission of the analog signal -- In this case, transmission of the AE signal from the field transducer/amplifier sensor is either by coaxial cable or transmitter to an interface and then to telephone line. The telephone line is tied directly to the base station for data collection and interpretation. The advantage is that no conversion of the monitored signal is required. Disadvantages are relatively high expense, analog signal transmission is subject to noise problems, and continuous transmission has high power requirements.

2. Transmission of analog signal only where signal level exceeds prethreshold level -- In addition to the previous setup, a threshold sensing circuit, consisting of a wideband discriminating amplifier and switching circuit, is necessary. Two alternates for transmission to telephone lines are again possible, i.e., direct coaxial cable and wideband FM transmitter and receiver. The trade-off of advantages and disadvantages over the previous method is one of lower power requirements versus slightly greater system complexity.

3. Transmission of warning signal if AE parameter exceeds a preset threshold -- Using a wideband discriminating amplifier and threshold discriminating circuitry, a CW transmitter can send a warning signal to a CW receiver at a base station. This transmitted signal generates a R.F. carrier with no modulation to the receiver which senses whether a signal is

present or not. Advantages are simple transmitter, low power consumption and high noise immunity. Disadvantages are no direct signal information, no false signal discrimination and no way of interrogating the monitoring location.

4. Transmission of warning signal if AE parameter exceeds a preset threshold, with provision to then transmit analog data upon radio command from base -- The equipment requirements are as with the previous case, plus a wideband FM transceiver at both the site and base station. The advantages are that only the needed data is transmitted, thereby reducing power requirements. False warning signals can also be discriminated and threshold parameters can be changed from the base station. The primary disadvantage is that wideband analog signals are still being transmitted in the transit mode.

5. Transmission of voltage level proportional to frequency -- In this configuration one needs a frequency to voltage converter and a narrow band FM transmitter or transceiver. The linkage from the monitoring site is the same as before except for substitution of a transmitter type. There are many advantages to this scheme which include: inexpensive voltage converter circuitry, low power consumption, higher noise immunity, ease of multiplexing of multiple pickup sites and any of the transmission modes described previously can be used. The disadvantages are that on-site data storage for later retrieval is difficult and noise immunity during transmission is not as great as with digital signals.

6. Digitizing and storing signals with transmission of data blocks on demand -- In this case, the equipment needed is the same as in No. 5 with the addition of an analog-to digital converter and a microprocessor and data storage medium with non-volatile memory or magnetic tape. The linkage to the transmission system is by CW transceiver or telephone interface (modern type). The advantages are that monitoring modes can be easily changed, data storage is easy, power consumption is low, high noise immunity exists, transceiver circuitry is simple, multiplexing is possible and data (in addition to the AE signal) can be stored on site. The disadvantages are that some software development is needed and the storage medium is delicate if tapes are used.

Summary and Conclusions

The acoustic emission (AE) monitoring of subsidence and subsidence related problems offers investigators two possible directions to follow. First, is for quantitative input into subsidence prediction models and the second, as an end in itself, gives an AE versus time response indicating when activity becomes intolerable. In this latter case, it is felt that AE precursors of instability occur much earlier than any type of deformation monitoring. This is due to the high sensitivity of the equipment and the relatively large spacial zone surrounding the wave guide.

In its simplest form, AE monitoring gives an AE count rate versus time which has been shown to correlate well with deformations taken at the same site, but do so in a much shorter time frame. This was illustrated in the first two situations presented. The second two situations illustrate use of the AE method as an on-line, continuous monitoring system.

When the land mass to be monitored becomes large, or relatively inaccessible, remote sensing becomes an attractive monitoring possibility. Here, six possible schemes were presented utilizing continuous and periodic transmission which included direct coupling, telephone and airborne methods.

In conclusion, it is felt that the AE method represents a fundamentally sound, straightforward and reasonably economic method to monitor a wide range of subsidence problems.

Acknowledgments

Research and development funds for AE monitoring of soil and rock masses has been provided for by the following organizations and project officers. Sincere thanks to all is expressed, and to the myriad of companies and students which aided in the progress of the work.

- Environmental Protection Agency; John E. Brugger
- Bureau of Mines; Frederick W. Leighton
- Federal Highway Administration; John R. Sallberg
- Army Research Office; Steven J. Mock (current)
- National Science Foundation; Charles A. Babendrier (current)

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THE SURVEY OF SOIL VERTICAL MOVEMENTS IN THE REGION OF BOLOGNA

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Abstract

The soil subsidence in Bologna area has been known for at least 30 years.In this paper after a short review of the known phenomenon we comment the planned project of high precision levelling network and some results of the first measurements just carried out with the purpose of checking subsidence.

General

It has been known for at least 30 years that the soil in the Bologna area is subsiding.The comparison between the height of some bench marks of Italian levelling network levelled in the period 1897-1902 and relevelled in the period 1947-1957 reveals this phenomenon.On the ground of these data Salvioni (1953,1957) was able to draw some contour lines of equal subsidence in the Po valley.The Bologna area showed a slight vertical sinking of the soil ranging from 10.7 cm to 20.9 cm in that period (fig. 1).

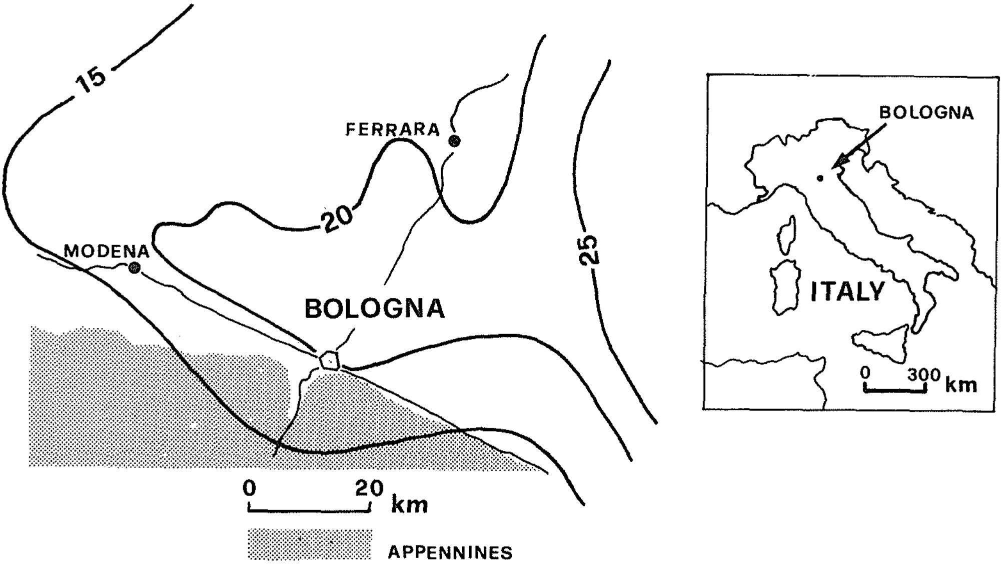

Fig.1 Contour lines in cm of equal subsidence in the period 1897-1957 (Salvioni,1957).

After 1950 there is a lack of information about the subsidence, however the phenomenon has become increasingly pronounced.

This fact has been brought out by the comparison of the geometric levelling carried out around 1943-50 and some repetitions performed by I.G.M.I. and other public organizations in the period ranging from 1970 to 1973.

The resulting soil movements and their distribution are set out in fig.2

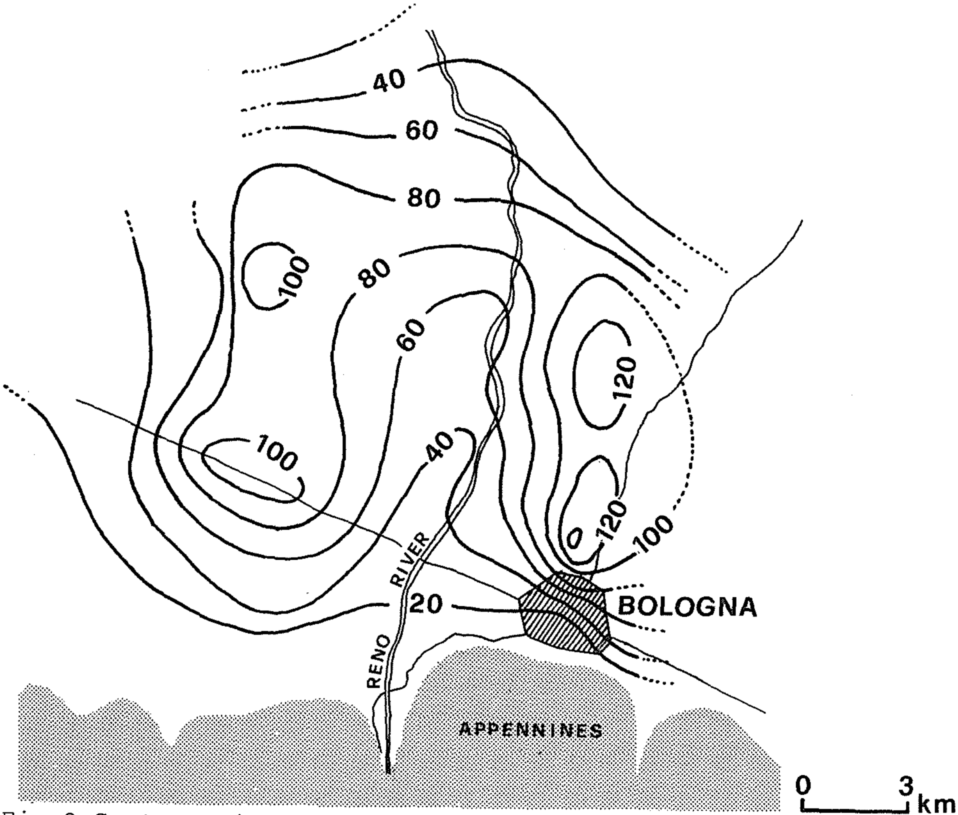

Fig.2 Contour lines in cm of equal susbidence in the period 1943/50 - 1970/73.

One can see between two characheristic bands with a sinking of the soil greater than 1.2 m a steady strip of land in relation to the river Reno. Contour lines converge in the centre of Bologna city at the foot of the hills. With the purpose of checking the phenomenon of vertical sinking of the soil the "Istituto di Topografia" of Bologna University working together with "Ufficio Tecnico del Comune di Bologna" in 1978 carried out two high precision levelling lines crossing the historical centre of the city and repeated the measurements

10 months later. The comparison of levellings, showed a sinkig of the soil greater than 6 cm/year.

In the same period people began to notice damage in some old buildings in the north Bologna area and an interesting phenomenon concerning the Asinelli tower was observed. Since 1970 we have been performing measurements to check the stability of the tower which is leaning and during the repetition of the measurements it was noticed that a new component in a northerly direction was added to the normal inclination.

In the period from 1974 to 1981 some public organizations have executed a great number of levelling lines. These data providing disconnected relevelled segments do not allow us to draw contour lines. Nevertheless these data brought up to date our knowledge of soil movements.

These measurements were carried out starting from different bench marks and at different time so that it was necessary to uniform the data. For this purpose we assumed as junction points bench mark settings in the more stable areas.

Following the advice of geologists we choose the segment between bench mark 186 and 163 of the line of I.G.M.I. n. 5 (Firenze-Bologna). In particular the bench mark 163 was used for connecting the heights of points belonging to different levelling lines. Fig.3 shows the vertical movements of bench marks along the main levelling lines of I.G.M.I. in the period 1973-1981.

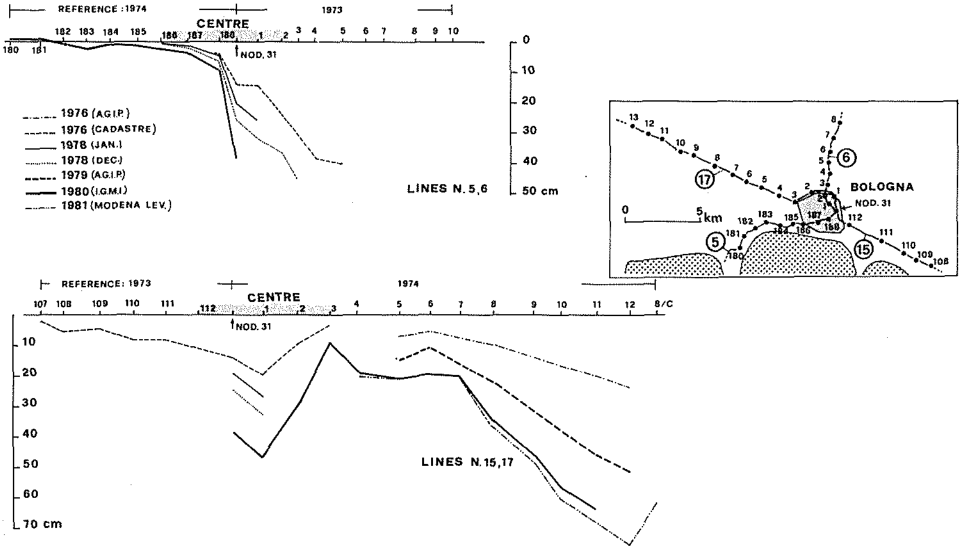

Fig.3 Vertical displacement profiles along levelling lines n.5,6 and n. 15, 17.

We analysed only the more urbanized area of Bologna. The graph 3.a shows that the area at the foot of the hills is little affected by the soil sinking; going to the north there is a sharp increase in the soil movement in the civic centre (bench mark 5/188 - Nodal 31) with a maximum sinking of 45.9 cm in the period 1973-1978 which, in the linear hypothesis, means a sinking velocity greater than 9 cm/year. The graph 3.b shows a gradual sinking of the soil, increasing from the east boundary to the centre of the town with a maximum of 47.5 cm in the period 1974-1980. In a westerly direction one meets an area almost stable to the river Reno and then one finds a gradual increas in the soil sinking with a maximum of 81.9 cm in the period 1974-1981 which, in the linear hypothesis, corresponds to a sinking soil velocity of 11.7 cm/year.

Considering the importance of soil movements and the probable consequences to the surrounding territory the technical office of Bologna municipality in the year 1981 commissioned some experts of Bologna University (Selli et al.,1981) to plan a study of this phenomenon. The planned program includes several subjects whose developement was expected within four years in order to know all the aspects of the soil sinking phenomenon. The execution of the programme began in 1982 with the institution of a levelling network. The measurements of the network have just been concluded and we are now going to elaborate the measurement data.

The area under control

The area to be checked is about 460 km^2 and includes the localities where the greatest soil sinking has taken place. First of all we planned to project a high precision levelling network utilizing only new bench marks, but in order to have information on the soil movements from the first measurements, we included in the network as many old bench marks as possible provided they were found to be stable. It has been possible to include 170 old bench marks in the network.

Reference bench mark

The problems concerning the vertical datum and the reference bench mark are closely connected with the purpose of the levelling network. All authors agree in considering the repeated levelling as the most reliable method for measuring vertical soil movements but,when a levelling network is not available it is possible to use scattered relevelled segments (Vanicek and Krakiwski,1982). In fact for a preliminary examination of the subsidence phenomenon we have also used this method even though in a rough way using the available scattered levelling data. Now we have projected and measured for the first time a levelling network the purpose of which is to detect vertical soil movements.

Concerning the reference bench mark it is pratically impossible to connect the Bologna control network with the Genova tide gauge which gives the mean sea level assumed as vertical datum for the Italian levelling network. In fact the levelling line Bologna-Genova is about 340 km long.

It is therefore clear that the best way is to locate some reference bench marks around the area to be checked .

Unfortunately almost all those areas are subsiding.

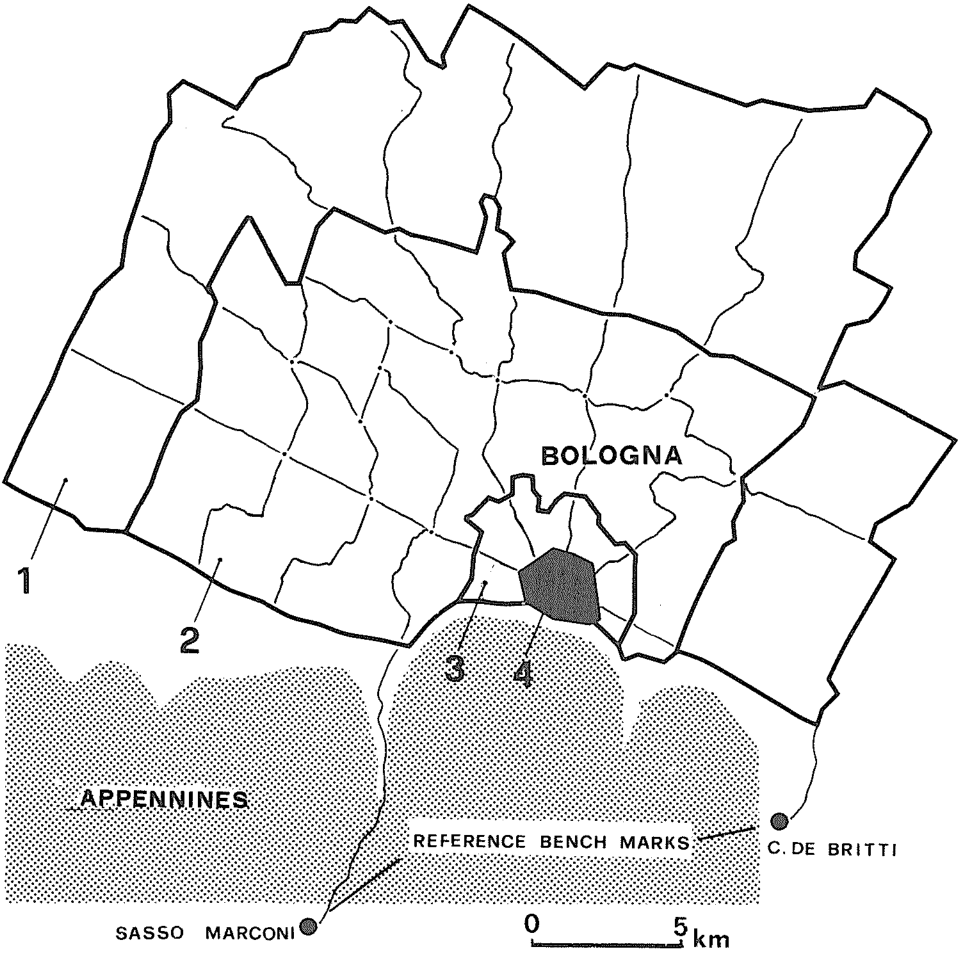

Fig.4 Levelling network with distances between consecutive and between nodal bench marks.Such distances are respectively: 1 km and 6.5 km in area n.1; 1 km and 3.5 km in area n.2; 0.5 km and 2 km in area n.3; 0.25 km and 0.5 km in area 4.

Nevertheless after a careful examination of levelling lines repeated at intervals of several years and after investigations on the spots carried out with the geologist prof. Mario Ciabatti, we have been able to find two bench marks that are almost stable: the one is located in "Sasso Marconi" area and the other in "Castel dei Britti" area. As a precaution againts damage, near each one we set up other bench marks on rock or on buildings with foundation resting on rock.

Layout of the levelling network

The levelling network has a lenght of about 375 km . It has been divided into four parts as regards the density of bench marks. Fig.4 shows the distances between consecutive and nodal bench marks in each area. In the center of Bologna we have the highest density of bench marks for detecting local soil movements in connection with the stability of buildings and with hydraulic problems.

The orientation of the levelling lines is as nearly parallel as possible to the direction of the greatest subsidence. For this reason for example the lines near the south side of the river Reno are almost perpendicular to the riverbed. Moreover the levelling lines avoid as far as possible the roads that can be changed in a short time. The bench marks, 475 in number, are located almost always on outer walls of old buildings. We rejected the new buildings or those which might be changed.

In fig. 5 the scheme of the levelling net covering the centre of Bologna is shown. It is easy to see the higher density of bench marks to the north where there are greater soil movements. In the same figure the areas are indicated in which we set up our local levelling networks with the purpose of studyng the movements of buildings or monuments.

The survey

The levelling measurements have been already carried out using instruments and methods of high precision levelling. Only a few technical lines have been measured with the purpose of connecting old scattered bench marks to the new levelling network. We established the values of discrepancies between forward and backward levelling of a section to be in modulus less than $T=3 \text{ mm} \sqrt{D}$ (D is the length of the section in km). When that discrepancy exceedes T the rejection test must be satisfied.According to this test a levelling x is rejected when

$$|x - \bar{x}| > 3.7 \text{ mm} \sqrt{D}$$

(where $\bar{x}$ is the mean of the separate means of the forward and backward levelling for the section).

In order to remove gross errors and to reduce the accumulation of systematic errors we stated rules concerning levelling.

procedures, instrumentation testing and rod calibration. We also laid down limit for the loops misclosures. As regard air refraction corrections we did not impose the collection of the atmospheric data because all the area of the levelling network is flat and the atmospheric corrections would be small and uncertain.

Since an influence has been shown of the earth's magnetism on automatic levelling instruments we recommended the use of automatic nonmagnetic levels.

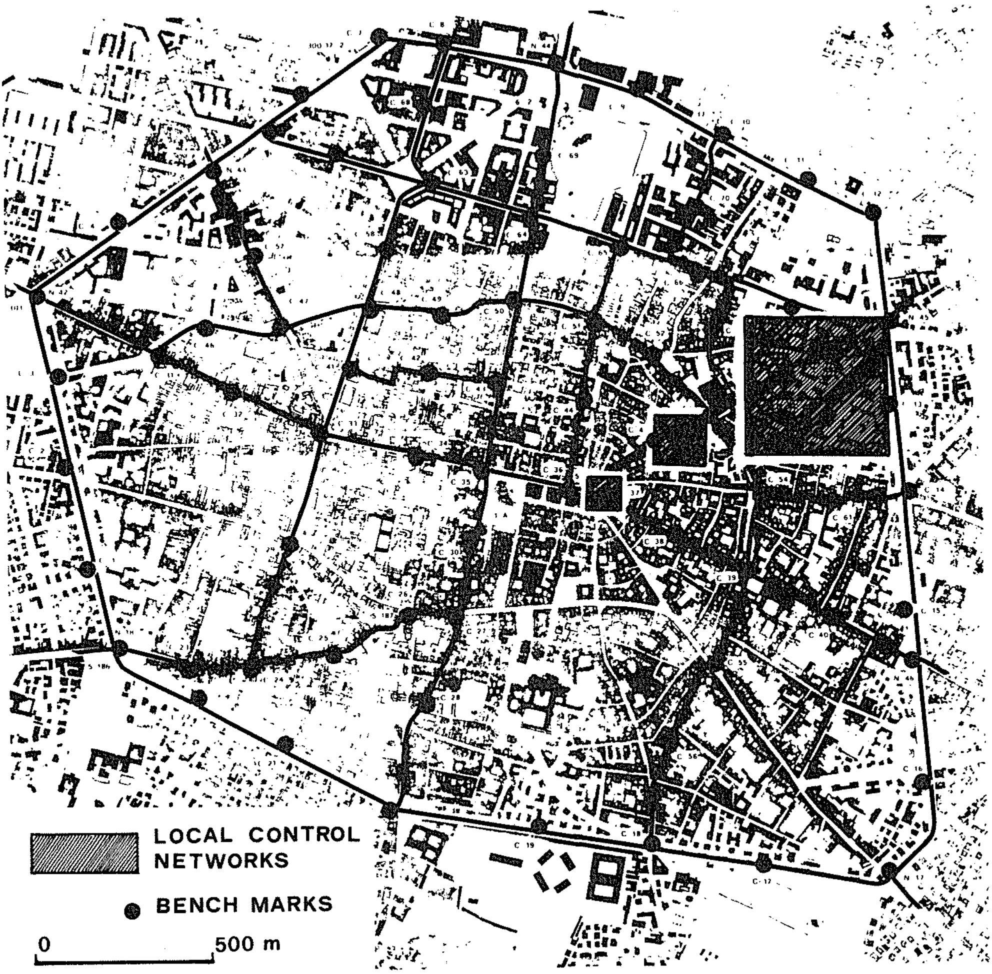

Fig. 5 Scheme of the levelling network of the centre of Bologna with the sites in which local control networks have been set up.

In the case of the Bologna levelling network the cause of the more dangerous systematic error is the considerable speed of subsidence. In fact during the measurement of the levelling network large soil movements can occur so that we need to reduce the levelling data to a common period. However we laid it down that the measurement period had to be at most 45 days. Moreover we indicated some levelling lines we called "criticals" to be measured first of all and in any case within 30 days (fig.6). Employing at the same time 7 levelling crews we completed this programme.

First results

Levelling network measurements have just been carried out; after the corrections due to the calibration rods and after the technical and administrative controls we will perform the adjustment of the network.

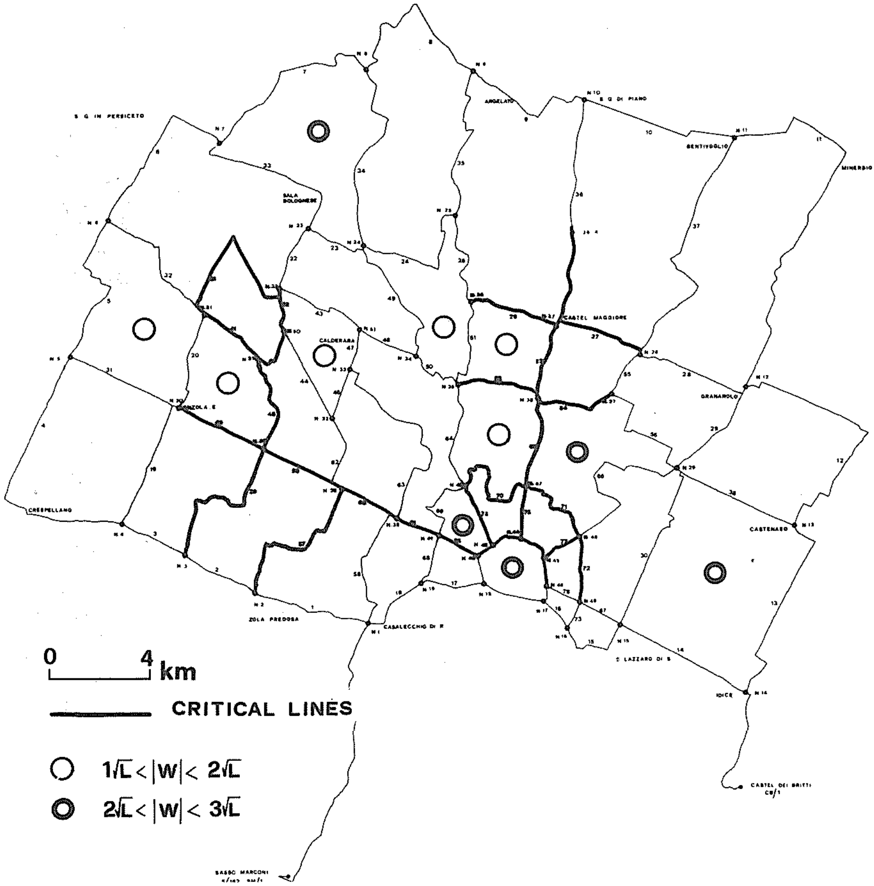

Fig. 6 Levelling loops misclosures W in mm and critical lines.

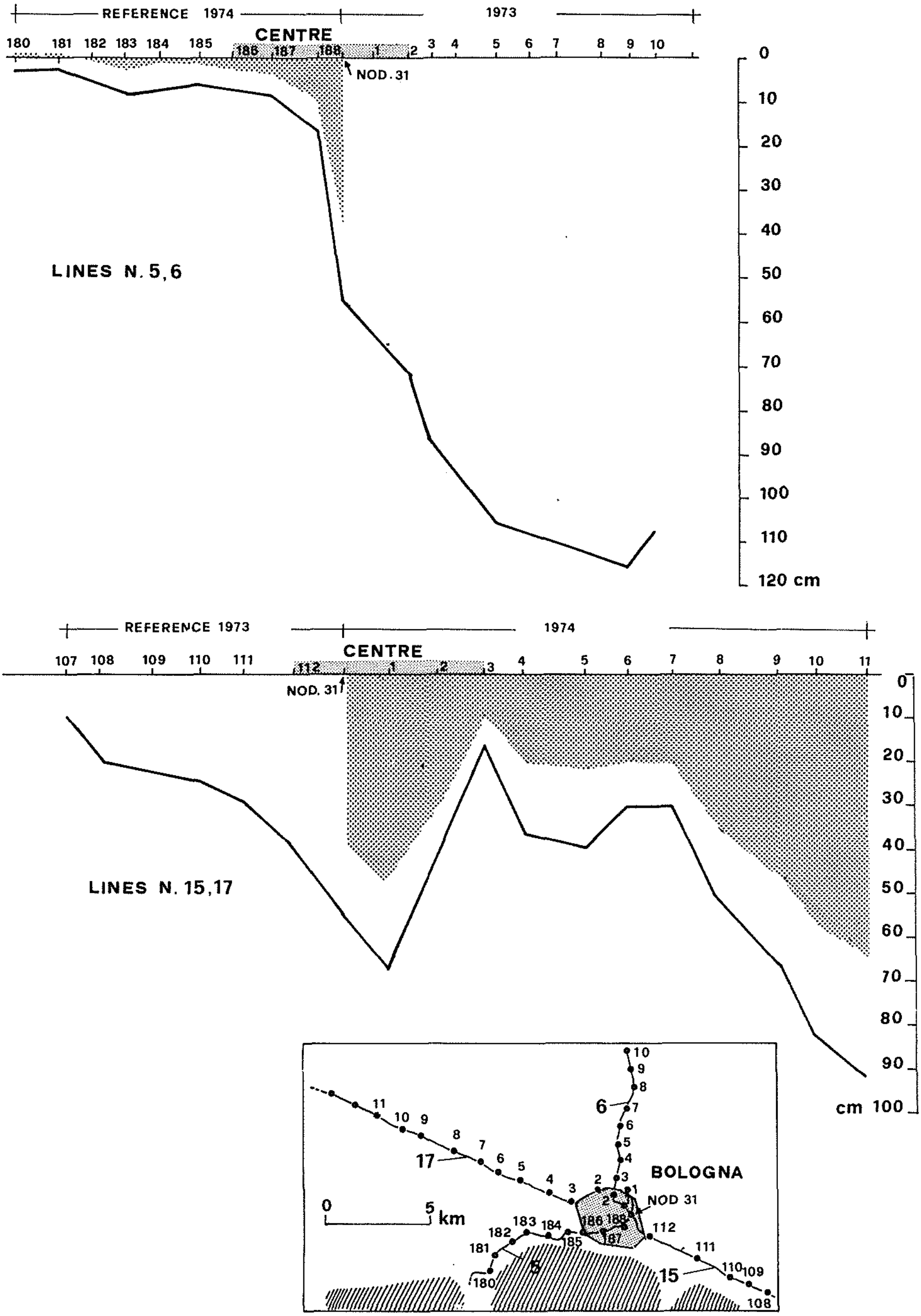

Fig. 7 Vertical displacements profiles along levelling lines n.5,6 and n. 15, 17.

The Bologna municipality authorized us to present some provisional results of the levelling concerning only the urban centre.

Fig.6 shows the loops in which misclosures exceed $\sqrt{L}$, L being the length in km of the loops. These errors are small and this means that also the adjustment corrections will be small. Therefore the provisional data of height differences we present here are not very different from those obtained after adjustement is carried out. In Fig. 7 we give the soil movements along the same levelling lines shown in fig.3. The grey area shows the soil movements which appeared from previous measurements.

The trend of the subsidence is confirmed, moreover we have new data relating to the northern area of the town in which the maximum subsidence (bench mark N.6/10) is found to be 116.3 cm in the period 1973-1983 which, in the linear hypothesis, corresponds to a sinking velocity of 11.6 cm/year. In Fig.8 we set out the sinking of two bench marks of great interest in the period ranging from 1950 till now.

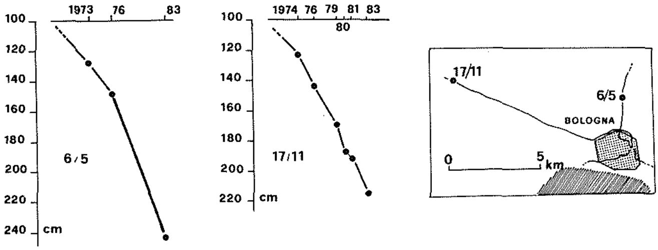

Fig.8 Diagrams of sinking of bench marks n.6/5 and n. 17/11 with respect to time.

Conclusions

The first results of measurements besides confirming the satisfactory layout of the survey, show that the phenomenon of subsidence in Bologna is proceeding with a disturbing velocity which has obtained 11.6 cm/year.

We hope to have, as soon as possible, the data resulting from the adjustment of the network in order to have an overall description of subsidence in the Bologna area.

We also urge the Bologna municipality to consider the absolute necessity of carrying out the programme elaborated in 1981 by the experts (Selli et al.,1981) in order to acquire knowledge of all the aspects of the subsidence phenomenon.

Acknowledgements

We have to thank Prof. M. Ciabatti who gave us geological support for the choice of stable bench marks and the land-surveyor G. Scuderi who gave us a great deal of help and has directed the levelling measurements.

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SOME NOTES ON THE REDEFINITION OF A VERTICAL REFERENCE NETWORK FOR THE STUDY OF THE SUBSIDENCE IN THE EASTERN PART OF PO RIVER VALLEY.

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Abstract

Many levelling data are available for the study of ground vertical movements in eastern Po river valley.Measurements have been carried out at various epochs assuming different reference bench marks which sometimes have been involved in subsidence phenomena.Unfortunately the determination of height variations of such bench marks with respect to the zero reference of the National Levelling Network is not possible.Moreover relative vertical movements between some reference bench marks are uncertain because of the the presence of incoherences in fundamental levellings.An investigation of single measurement campaigns points out possible causes of these incoherences. Research is therefore necessary in order to get a height reference system which allows levelling data concerning the region to be homogeneous; the research program is outlined.

Introduction

Numerous levelling lines and networks have been set up for different purposes and at various epochs in the eastern part of Po river valley. In fig.1 we report the lines of the I.G.M.I. (Istituto Geografico Militare Italiano) National Levelling Network and the most important local levelling networks established in order to study vertical ground movements.

Local networks were usually linked to I.G.M.I. bench marks whose heights had been determined by the I.G.M.I. itself in the period 1942-1952,in which the survey of the northern and central parts of the New Fundamental Height Network was carried out. The n.19 line Portomaggiore-Mestre only was relevelled in 1956 and in 1958-1959.

Till 1970 these heights were regarded as invariable in time,save in the region of Po river delta,where a considerable subsidence had been pointed out for some time (Salvioni,1953).

The I.G.M.I. and the Cadastre in the period 1968-1980 relevelled some levelling lines situated in Po river valley between Alps and Appennines. In fig.2 we set out the course of such lines,the years of all the surveys performed till now and the various reference bench marks used.

The comparison between the results of measurements carried out in the period 1942-1974 pointed out that remarkable ground sinkings had taken place in wide regions later than the period 1942-1952 (fig.3).

Because of this phenomenon the heights of some reference bench marks of local levelling networks changed in time. By consequence it appeared

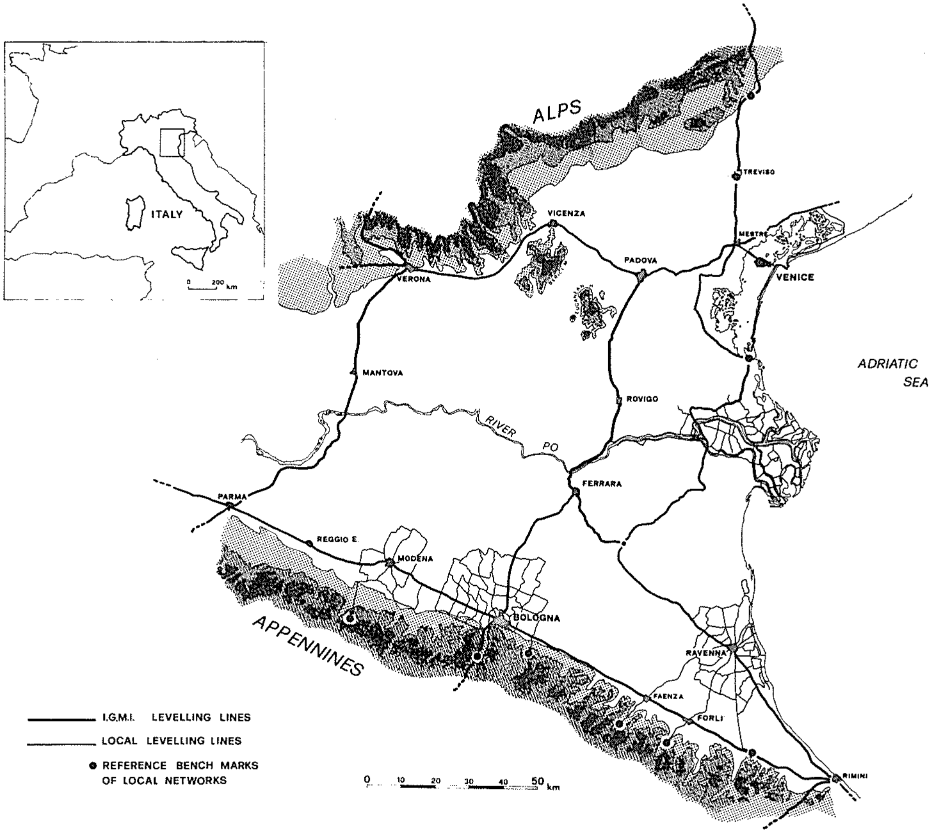

FIG.1 Most important levelling lines and networks in eastern Po river valley

necessary to determine the magnitude of height variations of such reference bench marks occurred between measurement campaigns in order to evaluate the ground sinking in any zone with respect to the same zero reference.

However the simple comparison between heights determined by the I.G.M.I. and by the Cadastre since 1942 is not sufficient to obtain movements of local reference bench marks,because the vertical datum used in the period 1942-1952 is probably different from the one used in the period 1968-1980. Between 1942 and 1952,in fact,heights were referred to the mean sea level (m.s.l.) recorded at Genova tide gauge in 1942,while between 1968 and 1980 heights were calculated with respect to five fundamental reference bench marks believed stable in time;heights values determined by I.G.M.I. in the period 1942-1952 were assumed for them.

This means that the temporal height variations of any bench mark in eastern Po river valley which result from the comparison of levellings carried out at different epochs indicate the real values of its vertical movements on condition that the heights of the above mentioned fundamental

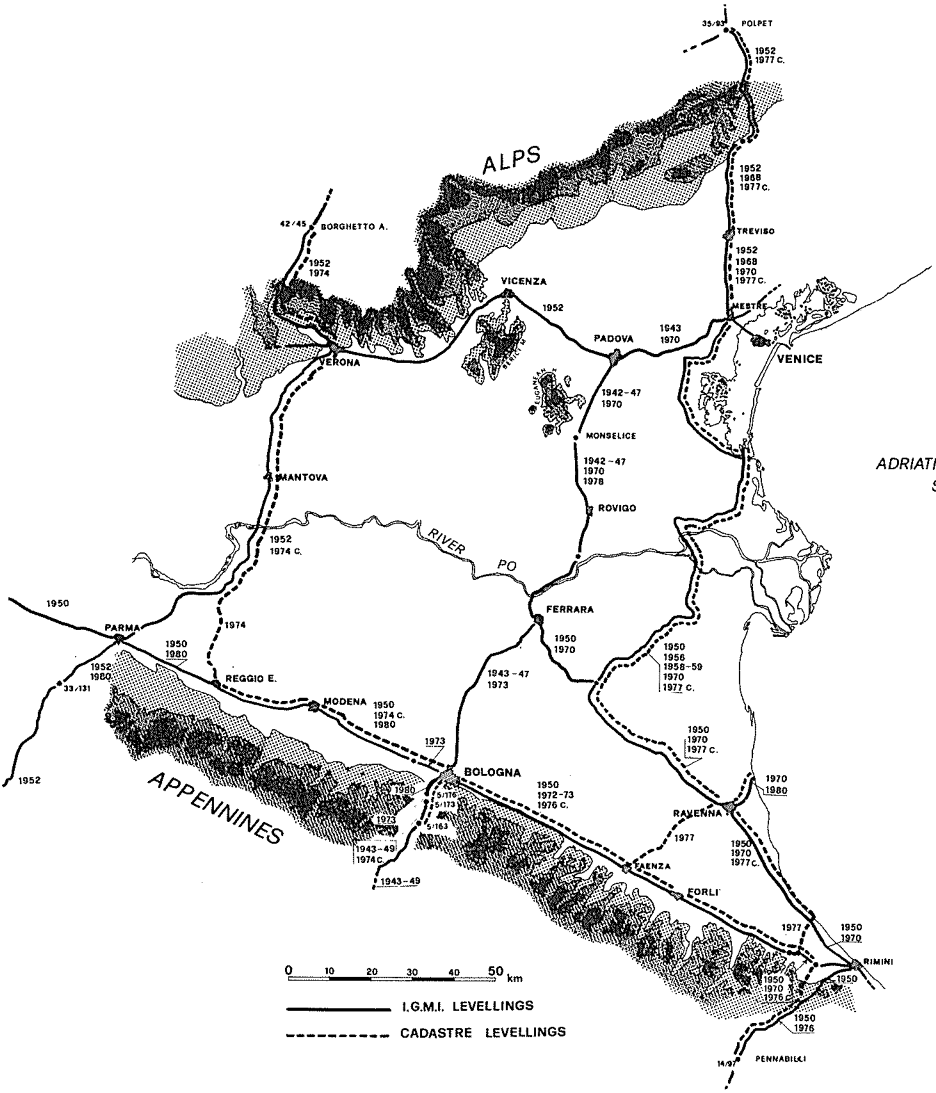

FIG.2 Levelling carried out by the I.G.M.I. and by the Cadastre in the period 1942-1980

reference bench marks have not changed in time with respect to 1942 Genova mean sea level.

Unfortunately the differences of height between such reference bench marks and Genova tide gauge have not been measured again after 1942-1952, whereas levelling lines between the reference bench marks have been relevelled,even if at different times.By consequence available data up to

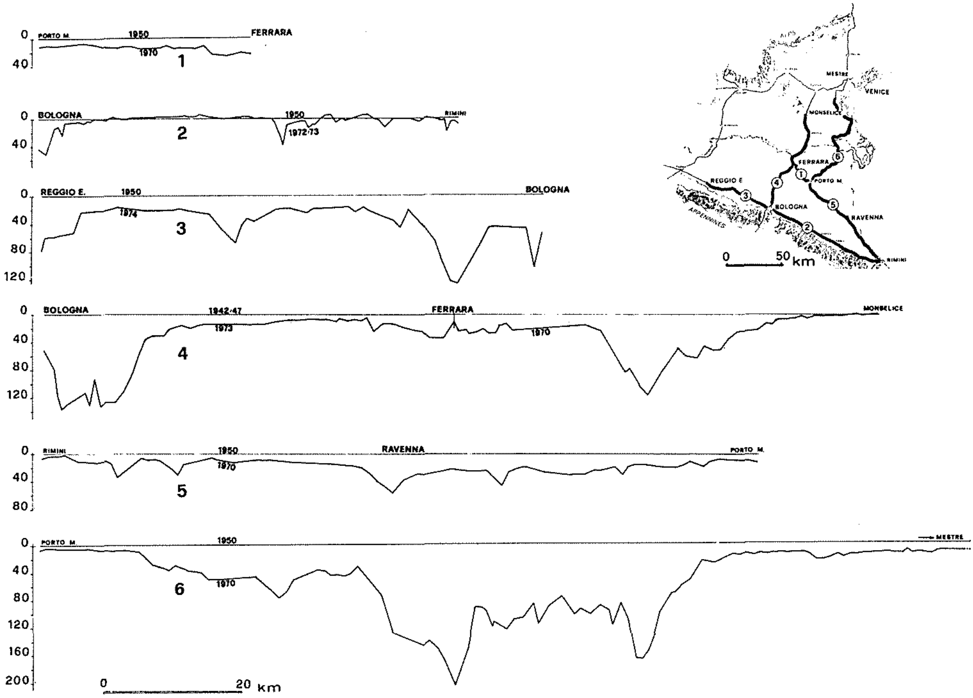

FIG.3 Displacement profiles in cm along some I.G.M.I. levelling lines in the period 1942-1974

now allow us only to check the eventual invariability of height differences between the reference bench marks themselves.

Anyhow such a study is remarkably important for the research about the subsidence in eastern Po river valley. In fact,if the mentioned differences have not changed since 1942-1952,then it is possible to refer the heights of any bench mark existing in the valley to a common regional reference system constituted by these five reference bench marks.In this system ground movements would be determined too.

Analysis of bench marks stability

First of all we have examined height temporal variations of bench marks set up in zones where significant relative ground movements do not appear. Only official data have been considered.

Some irregularities have been detected in the movements concerning bench marks situated at the base of Appennines between the towns of Bologna and Rimini. In fig.4 and in fig.5 we have reported the following data: scheme of levelling lines levelled by the I.G.M.I. and by the Cadastre in the period 1970-1976,reference bench marks used and years in which their heights were determined,displacement profiles concerning sections of levelling lines

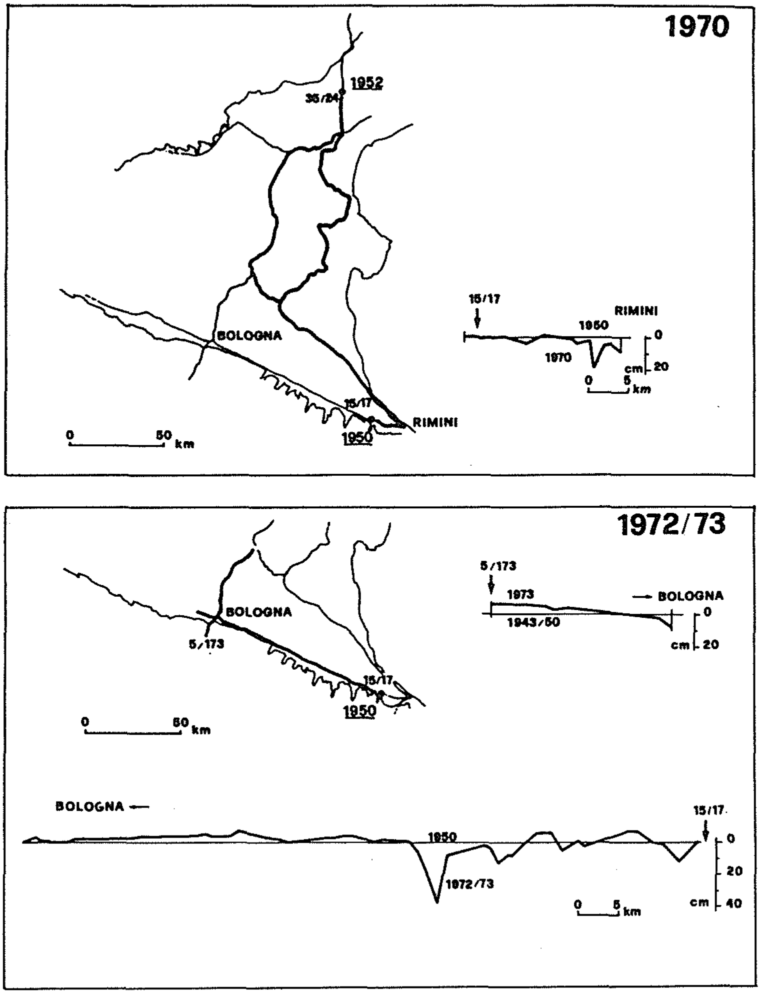

FIG.4 I.G.M.I. levellings carried out in 1970 and in 1972/73 and displacement profiles with respect to 1943/50 concerning some sections

obtained by the comparison between the results of surveys performed in different years as specified in every graph.

We note that levellings of 1972/73 point out that considerable rises in ground height occurred in Appennines near Bologna city (maximum value: 6 cm, bench mark n.5/173) and between Bologna and Rimini with respect to 1943/50; moreover it resulted that the bench mark 15/17 was stable in the period 1950-1970; levellings carried out in 1974 and 1976 instead indicate a ground sinking in the same zones in the period 1943/50-1974 and 1972/73-1976. In particular the height of the bench mark n. 15/17 (reference point in 1972/73

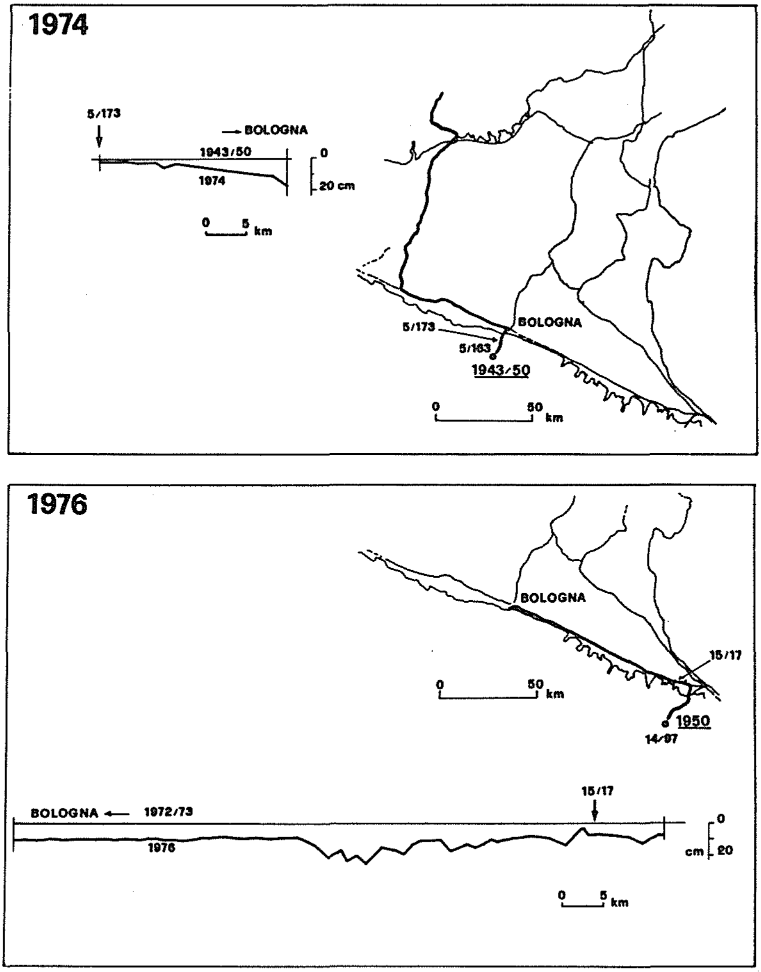

FIG.5 Cadastre levellings carried out in 1974 and 1976 and displacement profiles in the periods 1943/50-1974 and 1972/73-1976 concerning some sections

I.G.M.I. levellings) resulted 5.4 cm inferior to the value assumed in 1972/73 by the I.G.M.I.

The diagrams time/elevation of three bench marks located in the area under investigation are reported in fig.6.

Standard deviations of heights have been computed with regard to the different reference bench marks assumed, stating a standard deviation of 1 mm for a levelled height difference along a distance of 1 km. Diagrams of fig.4, fig.5 and fig.6 would point out that in some sites ground had rised in the period 1943/50-1972/73 and then has sunk. It is slightly probable that

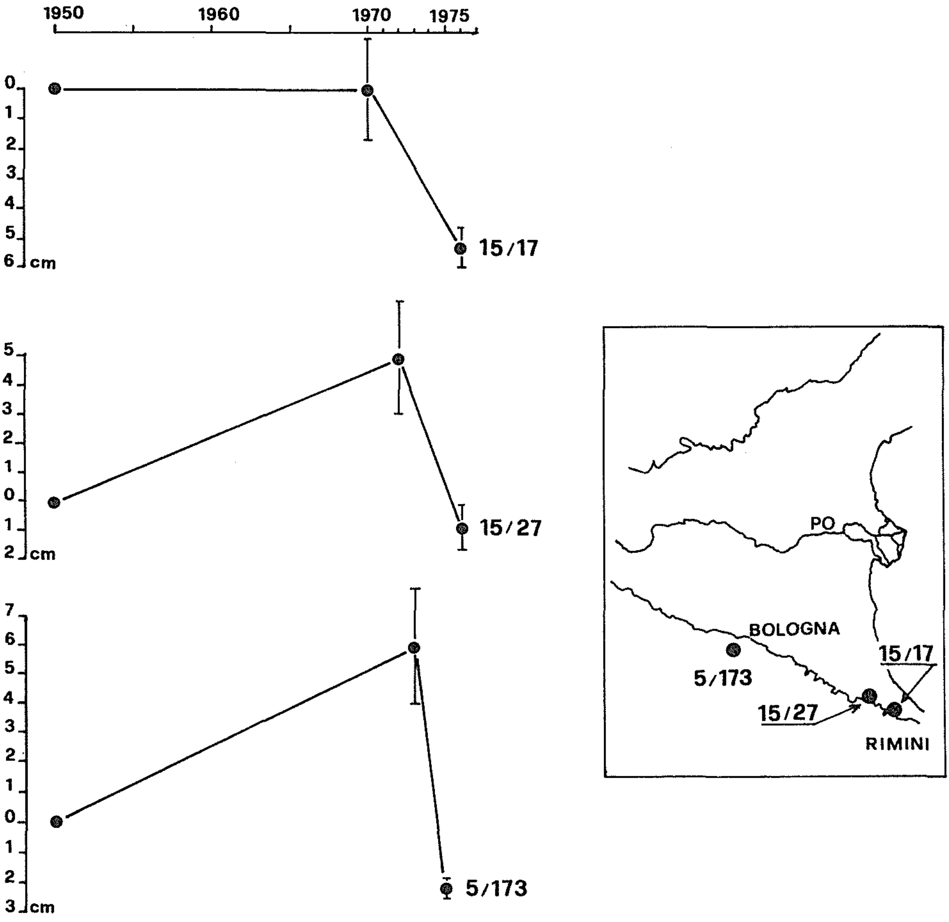

FIG.6 Diagrams time/elevation in the period 1950-1976 of the I.G.M.I. bench marks n.15/17,15/27 and 5/173.

such phenomenon has occurred in a region in which the general trend is toward subsidence.

Therefore the most believeble hypotesis is the presence of incoherences in levellig data.The possibility of a sinking of bench mark n.15/17 between 1970 and 1972 has been rejected because data indicate that a whole section long 5 km at least,considered stable in the period 1950-1970 by the I.G.M.I., has lowered.

Exam of single levellings

Having rejected the most elementary hypotesis,we have then controlled which reasons for incoherences are present in single levellings carried out in the period 1942-1980.

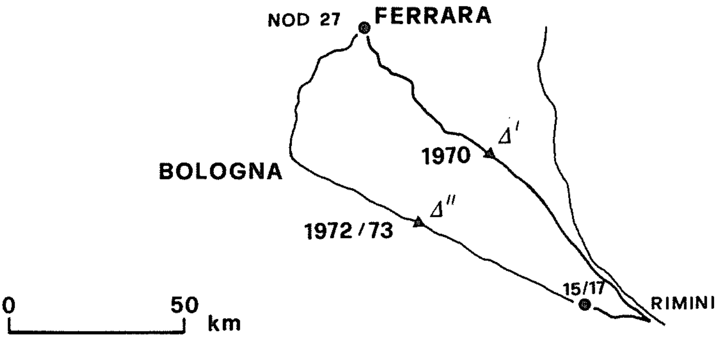

FIG.7 We give here an example of a possible incoherence in levellings. The height difference between bench marks Nod.27 (Ferrara) and 15/17 (Savignano sul Rubicone,Rimini) was measured by the I.G.M.I. in 1970 and in 1972/73 following different levelling lines;obtained values were respectively Δ'= 28.0939 m and Δ''= 28.0731 m. Therefore the loop misclosure 0.0208 m was acceptable with regard to the length of the loop and the accuracy of measures. However the comparison between levellings performed by the I.G.M.I. in 1950 and in 1970 indicated that the bench mark 15/17 was stable while the bench mark Nod.27 had sunk with an average velocity of 1.2 cm/year;then,measure errors excepted,it would have been more probable that Δ'' resulted 2-3 cm greater than Δ'. This means that the difference between measured and expected value of Δ'' is about 5-6 cm,which is a significant quantity.

The most important results of this study are outlined as follows:

- fields operations were usually protracted for months or years also in regions where considerable ground vertical movements were contemporaneously taking place;this fact has altered the homogeneity of levelled differences of elevation in the same measurement campaign (fig.7);
- levelling lines were often surveyed as branch lines; therefore the possibility of detecting blunders was reduced and no adjustment could be made;
- levelled differences of height were used in the adjustment of the I.G.M.I. networks in 1942-52 and in 1970-73 instead of GPU's (geopotential units) or dynamic,orthometric or normal height differences. Moreover the adjustment of the National Levelling Network was made dividing the network itself into two blocks: Central Italy network was previously adjusted with respect to the zero reference of 1942 Genova m.s.l.; then Northern Italy network was adjusted assuming the heights of some junction points,determined in first computation,as invariable (Salvioni,1957). The National Network was later

adjusted using GPU's (Ehrnsperger,1979); however the heights obtained in this calculus do not yet figure on official catalogues.

Research program

Now we have pointed out some incoherences in fundamental levellings and their probable causes. The first consequence of this situation is the indetermination of height corrections which make levellings linked to I.G.M.I. or Cadastre bench marks homogeneous.

This problem does not concern only old levellings,but also new ones.

Eastern Po River valley is,in fact,a wide region where several public organisations act with different aims. It is therefore probable that also in levellings which are going to be carried out different reference bench marks will be assumed,with regard to the various purposes of surveys and to the location of levelling lines or networks. In the study of local subsidence phenomena,for instance,the zero reference of heights may be arbitrary, provided reference bench marks are set up in stable areas.

In order to avoid new height incoherences,we think that several bench marks,whose heights are known at any epoch,should be available all over the territory. Moreover the most opportune zero reference for them is the mean level of Adriatic sea. Such vertical datum would,in fact,allow the study of very important technical problems concerning,for example,the reclaimed lands and the phenomenon of coast line withdrawing,besides the analysis of the subsidence in the region.

A height reference system which answers these aims can be fulfilled,but it requires that following studies are carried out:

- further analysis of all levelling data concerning the measurement campaigns performed by the I.G.M.I.,the Cadastre and other public organisations,in order to remove eventual undetected gross errors.
- evaluation of crustal movements by means of the study of the temporal variations of height differences between bench marks located in zones which have not been involved in local subsidence phenomena;
- reduction to a same epoch of height differences between the end points of the levelling lines which link the above mentioned zones;
- readjustment of the network constituted by such lines using height differences corrected for the influence of the earth gravity field and assuming as zero reference the mean sea level recorded by available tide gauges at the designed epoch;

When the otlined research will be over,we would have at our disposal:

- a certain number of zones,everyone more stable than the surrounding territory,in which bench marks of known heights with respect to mean sea level at the above mentioned epoch are located;such bench marks should be regarded as reference points;
- an opportune model concerning these zones,which allows us to determine,at any epoch,the height corrections for reference bench marks due to crustal movements;eustatic variations of sea level should be considered too.

Finally we think it opportune that the height differences between reference bench marks are relevelled at regular intervals.

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APPLICATION OF HIGH PRECISION LEVELLING AND PHOTOGRAMMETRY TO THE DETECTION OF THE MOVEMENTS OF AN ARCHITECTONIC COMPLEX PRODUCED BY SUBSIDENCE IN THE TOWN OF BOLOGNA.

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Abstract

The most remarkable effects of the subsidence of Bologna region have occurred in the north-east part of the historical centre of the city. Here the monumental complex of St. James Church and of G. B. Martini Conservatoire presents serious damages which are getting worse every year.

A high precision levelling network has been established in 1979 and measured six times (1979-1983) to detect the vertical movements of the structures and of the surrounding ground.

Moreover in the period 1979-1982 the deformations of the most seriously damaged part of the complex have been controlled by the photogrammetric method.

The results of the surveys show that the movements of the monumental complex are considerable and strictly related to the phenomenon of subsidence.

Introduction

St. James Church is one of the most important historical buildings of the town of Bologna; it is situated in Zamboni Street, at about 250 m far from the famous Asinelli Tower.

Both Gothic and Renaissance architectonic elements are characteristic of the Church, which preserves in the interior the famous Bentivoglio Chapel. An elegant arcade of XVth century runs along the left side, while on the right side there is the seat of the Conservatoire G. B. Martini, which keeps one of the most important music library in Europe.

High precision levelling

The structures of the Church and of the Conservatoire have begun to reveal damages several years ago. In 1979 it was thought it convenient to set up a height control network in order to detect the temporal evolution of the vertical movements of the two buildings. One of the purposes of the survey, in particular, was the check of the correlation between such movements and the subsidence phenomenon which has taken place for many years in Bologna city (Pieri and Russo, 1977) and whose effects had been noticed in other historical buildings (Borgia et al., 1977).

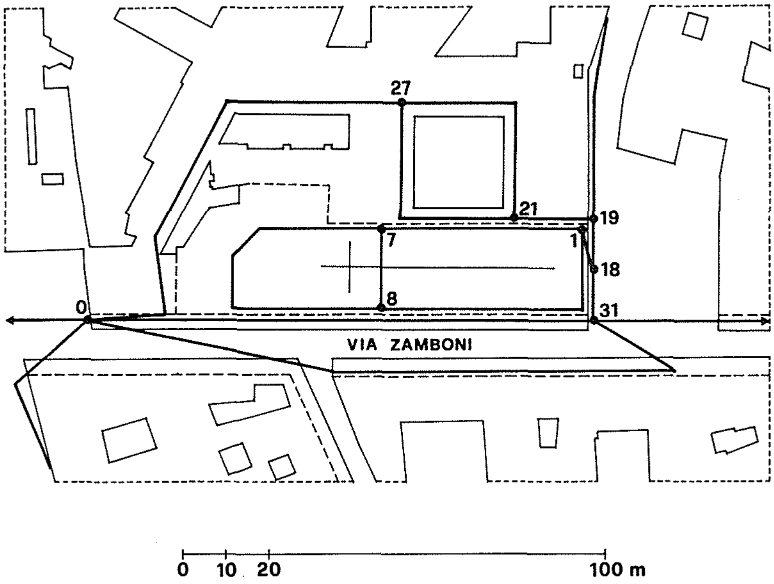

FIG. 1. Scheme of the height control network.

Measurements have been carried out adopting procedures and equipment of high precision levelling. The scheme of the network (FIG. 1) is constituted by three close loops outside the Church and by two close loops inside, joined by a short levelling line; four branch lines link some bench marks (b.m.) located in the surrounding area to the network.

Six measurement campaigns have been performed up to now: May 1979, November 1979, May 1980, November 1980, June 1982 and December 1983. In every set of measurements loops misclosures and discrepancies between forward and backward levelling of branch lines have not exceeded the limit $t = \pm 0.3 \text{ mm} \sqrt{n}$, where n is the number of level stations.

The network has been adjusted by least squares method and the heights of bench marks have been computed with respect to the height of b.m. 188/5 of the national levelling network: their values in m are listed in TAB. 1.

The heights of b.m. 188/5 at various epochs have been obtained by linear interpolation between the value of 59.0372 m determined by Cadastre in June 1974 and the value of 58.8643 m determined by Bologna Municipality in November 1983. Both levellings have been referred to the b.m. 162"/5, located near Sasso Marconi, in a site which has always been regarded as stable. Levellings performed by Cadastre in 1976 and by I.G.M.I. in 1980 have not been considered because the dates of

	May 1979	Nov. 1979	May 1980	Nov. 1980	June 1982	Dec. 1983
188/5	58.9539	58.9454	58.9369	58.9284	58.8883	58.8629
Nod.31	51.28372	51.24850	51.22085	51.19320	51.11580	51.04735
0	52.94446	52.91901	52.89668	52.87534	52.81055	52.75470
1	57.28096	57.26617	57.25334	57.23996	57.19850	57.16285
2	57.20707	57.19174	57.17880	57.16517	57.12343	57.08760
3	57.30156	57.28563	57.27180	57.25762	57.21323	57.17538
4	57.23758	57.22136	57.20741	57.19286	57.14847	57.10998
5	57.29125	57.27395	57.25916	57.24380	57.19662	57.15583
6	57.25020	57.23257	57.21757	57.20193	57.15449	57.11315
7	57.25721	57.23841	57.22236	57.20585	57.15535	57.11153
8	57.28961	57.27043	57.25411	57.23727	57.18625	57.14165
9	57.52270	57.50189	57.48429	57.46627	57.41222	57.36462
10	57.30941	57.28861	57.27107	57.25289	57.19903	57.15119
11	57.52912	57.50726	57.48872	57.46990	57.41349	57.36389
12	57.55352	57.53048	57.51118	57.49144	57.43309	57.38125
13	57.53001	57.50773	57.48871	57.46967	57.41196	57.36169
14	57.52529	57.50311	57.48421	57.46527	57.40783	57.35768
15	57.46215	57.44060	57.42234	57.40372	57.34788	57.29855
16	57.44227	57.42188	57.40460	57.38691	57.33343	57.28635
17	57.26607	57.24567	57.22826	57.21056	57.15700	57.10974
18	55.09676	55.08200	55.06894	55.05591	55.01445	54.97953
19	55.30942	55.29508	55.28242	55.26965	55.22872	55.19430
20	55.50904	55.49387	55.48012	55.46676	55.42348	
21	55.48012	55.46472	55.45024	55.43676	55.39235	
22	55.47163	55.45602	55.44141	55.42771	55.38256	

TAB. 1. Heights of bench marks in m.

measurements are uncertain and the heights are referred to different bench marks. However the results of these levellings, though slightly uncertain, confirm the linear trend of the sinking of b.m. 188/5.

In interpolation calculus a tampering of above mentioned b.m. occurred in 1981 owing to road works has been considered. Measuring the height difference variation between such b.m.

	May 1979	Nov. 1979	May 1980	Nov. 1980	June 1982	Dec. 1983
23	55.44466	55.42862	55.41364	55.39963	55.35334	
24	55.41816	55.40138	55.38616	55.37168	55.32407	
25	55.41427	55.39706	55.38138	55.36654	55.31793	
26	55.40144	55.38392	55.36811	55.35291	55.30345	
27	55.42254	55.40592	55.39089	55.37624	55.32848	
28	55.44130	55.42584	55.41164	55.39779	55.35248	
29	55.47827	55.46352	55.44971	55.43677	55.39351	
30	55.46757	55.45249	55.43833	55.42522	55.38137	
31	55.04405	55.02948	55.01621	55.00329	54.96110	54.92611
32	54.96815	54.95343	54.94013	54.92701	54.88496	54.85000
33	54.85428	54.83778	54.82363	54.80916	54.76470	54.72681
34	54.80631	54.78818	54.77305	54.75744	54.70933	54.66883
35	54.78216	54.76293	54.74615	54.72949	54.67812	54.63437
36	54.74887	54.72808	54.70995	54.69240	54.63784	54.59059
37	54.65010	54.62806	54.60872	54.59037	54.53289	54.48330
38	54.42442	54.40068	54.38022	54.36014	54.29921	
39	54.91146	54.89035	54.87158	54.85346	54.79706	
40	55.27824	55.26010	55.24369	55.22805	55.17730	
41	55.79771	55.78360	55.77124	55.75857	55.71787	55.68385
42	56.50816	56.49501	56.48348	56.47142	56.43326	56.40142
43	53.10229	53.07496	53.05110	53.02781	52.95993	
44		55.85417	55.84211	55.82969	55.79100	
45		55.29247	55.27920	55.26599	55.22378	
46		55.26563	55.25248	55.23889	55.19631	55.16015
47		55.04874	55.03128	55.01306	54.95966	54.91386

TAB. 1. Continued.

and another one, situated in close proximity, a movement $\Delta H = -0.0133$ m was detected.

Data listed in TAB. 1 indicate that the trend of bench marks sinking is quite linear. This is clearly pointed out in FIG. 2 by the diagrams time/subsidence of three bench marks (1, 12, 34) selected at random, whose positions are indicated in FIG. 3. A slight attenuation of the speed of movements can be noticed.

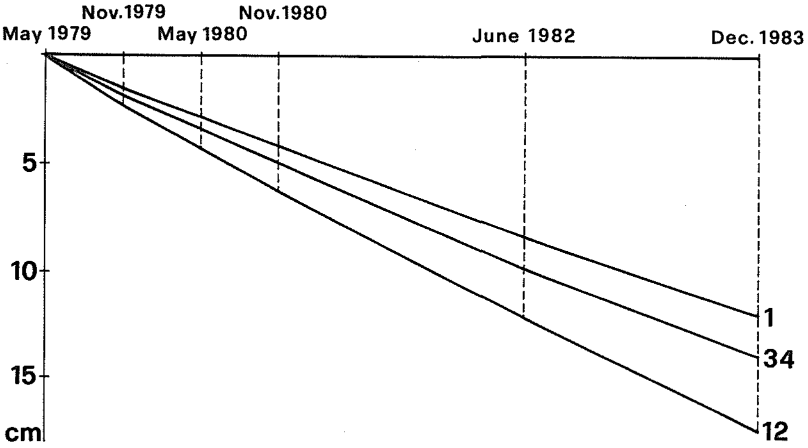

FIG. 2. Diagrams time/subsidence of three bench marks selected at random.

188/5	- 1.695	8	- 3.259	18	- 2.615	28	- 2.840	38	- 3.928
Nod.31	- 5.137	9	- 3.471	19	- 2.569	29	- 2.710	39	- 3.611
0	- 4.198	10	- 3.468	20	- 2.725	30	- 2.753	40	- 3.205
1	- 2.619	11	- 3.630	21	- 2.801	31	- 2.647	41	- 2.544
2	- 2.644	12	- 3.770	22	- 2.844	32	- 2.650	42	- 2.390
3	- 2.803	13	- 3.707	23	- 2.914	33	- 2.829	43	- 4.453
4	- 2 822	14	- 3.688	24	- 2.992	34	- 3.052	44	- 2.445
5	- 2.993	15	- 3.589	25	- 3.063	35	- 3.283	45	- 2.659
6	- 3.022	16	- 3.424	26	- 3.115	36	- 3.493	46	- 2.683
7	- 3.215	17	- 3.432	27	- 2.998	37	- 3.684	47	- 3.448

TAB. 2. Average annual movements of bench marks in cm.

Average annual movements of bench marks are reported in cm in TAB. 2. In order to have homogeneous values we have preferred to compute such movements for all bench marks in the period ranging from November 1979 to June 1982, because some heights were not determined in the first and last campaign. Values obtained are almost equal to the average ones computed with regard to the whole period May 1979 - December 1983.

Contour lines of equal speed of soil sinking in cm/year are reported in FIG. 3. The substantial regularity of lines points out that the phenomenon does not regard only the architectonic complex. This fact confirms that the damages of the buildings

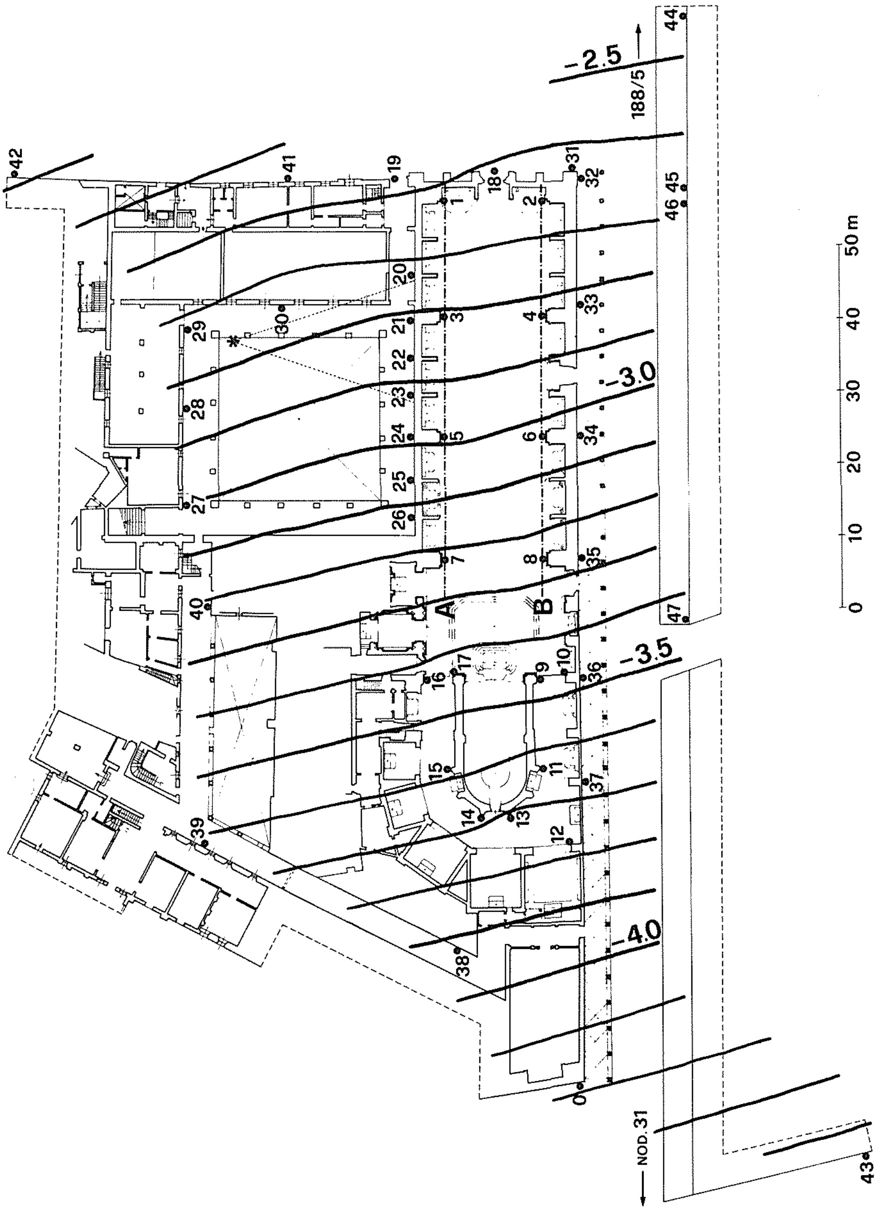

FIG. 3. Contour lines of equal speed of soil sinking in cm/year.

under control are produced by the subsidence phenomenon, which involves the whole urban area, and not by a local ground sinking.

The diagrams of the displacements of b.m. 3, 5, 7 with respect to b.m. 1 (FIG.3 Sect.A) and of b.m. 4, 6, 8 with respect to b.m. 2

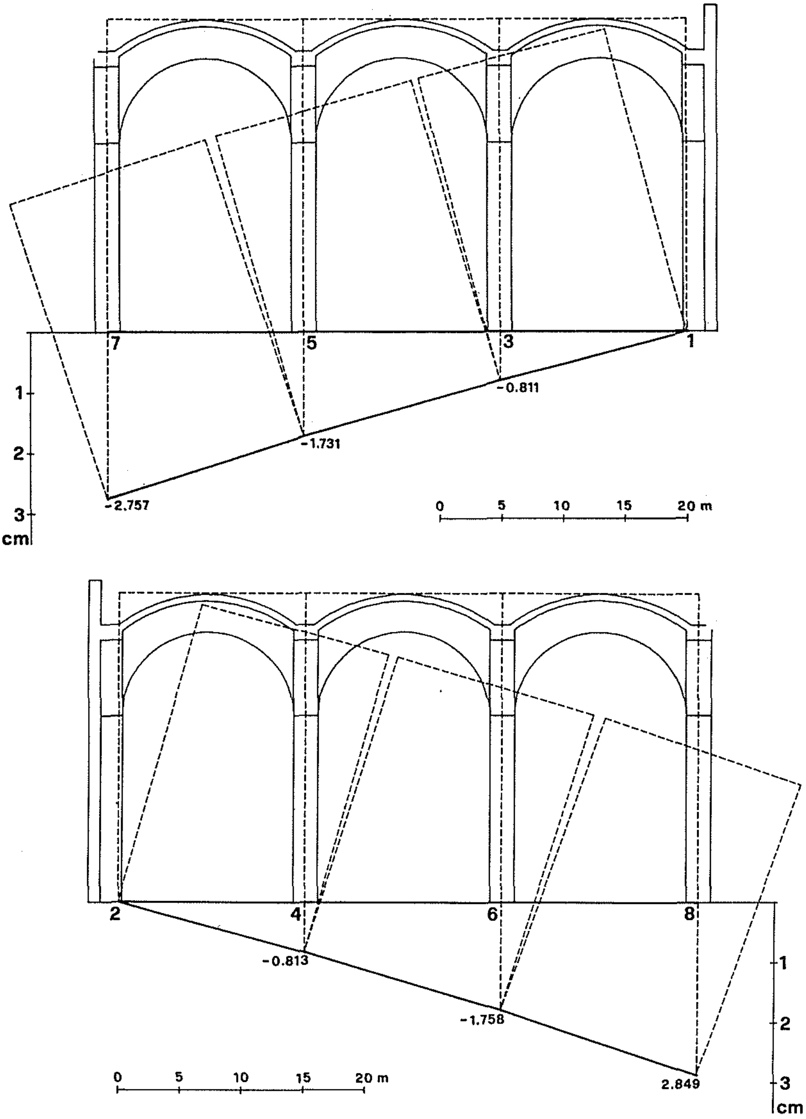

FIG. 4. Differential sinkings of the Church foundations.

(FIG.3 Sect.B) in the period ranging from May 1979 to December 1983 are reported in FIG. 4. The diagrams point out that the differential sinking of the Church foundations induces a state of stress in upper structures with a consequent possibility of damages and chains breaking, as it has really happened.

Now we wonder: why the architectonic complex of St. James Church - Conservatoire seems to have suffered from subsidence

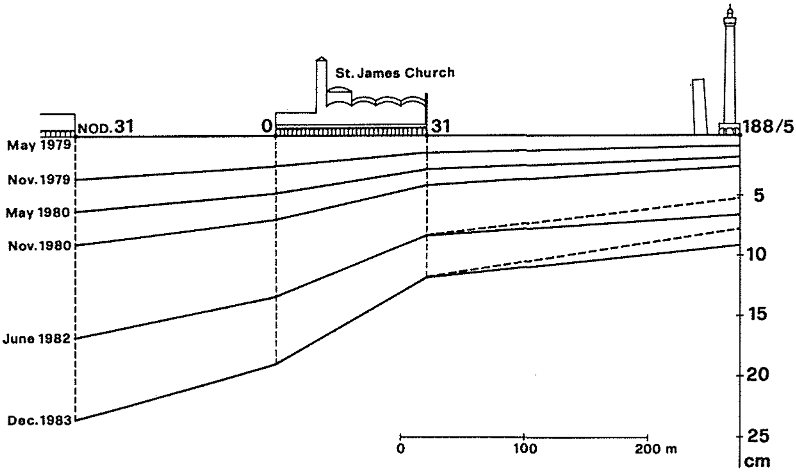

FIG. 5. The subsidence along Zamboni Street.

much more than other buildings situated in the same Zamboni Street? The large dimensions of the complex, above all along the direction perpendicular to the contour lines of equal sinking velocity, give without doubt an explanation of this fact. Moreover if we examine the FIG. 5, which represents a section along Zamboni Street, we observe that the Church is located just in the zone in which the soil sinking gradient appears to be more remarkable. This fact may be probably due to a break in continuity of underground, but the most interesting phenomenon with regard to the structures statics is the sensible variation of the soil sinking gradient connected with the positions of b.m. 0 and b.m. 31.

The discontinuity of gradient at the ends of the Church can be also noticed in FIG. 3 and above all in FIG. 4.

Dotted lines have been obtained under the assumption that b.m. 188/5 has not been tampered and then they indicate the real movement of the ground.

Photogrammetric survey

It has been carried out with the purpose of studying the evolution of some significant damages of the masonry structures of the complex. The wall under control, visible in FIG. 3, is both of the Church and of the Conservatoire cloister and it presented symptoms of anomalous stresses from the beginning of measurement campaigns. We have thought it interesting to detect the spatial movements of this wall, which appeared so remarkably stressed.

In order to verify the behaviour of a vertical strip of the

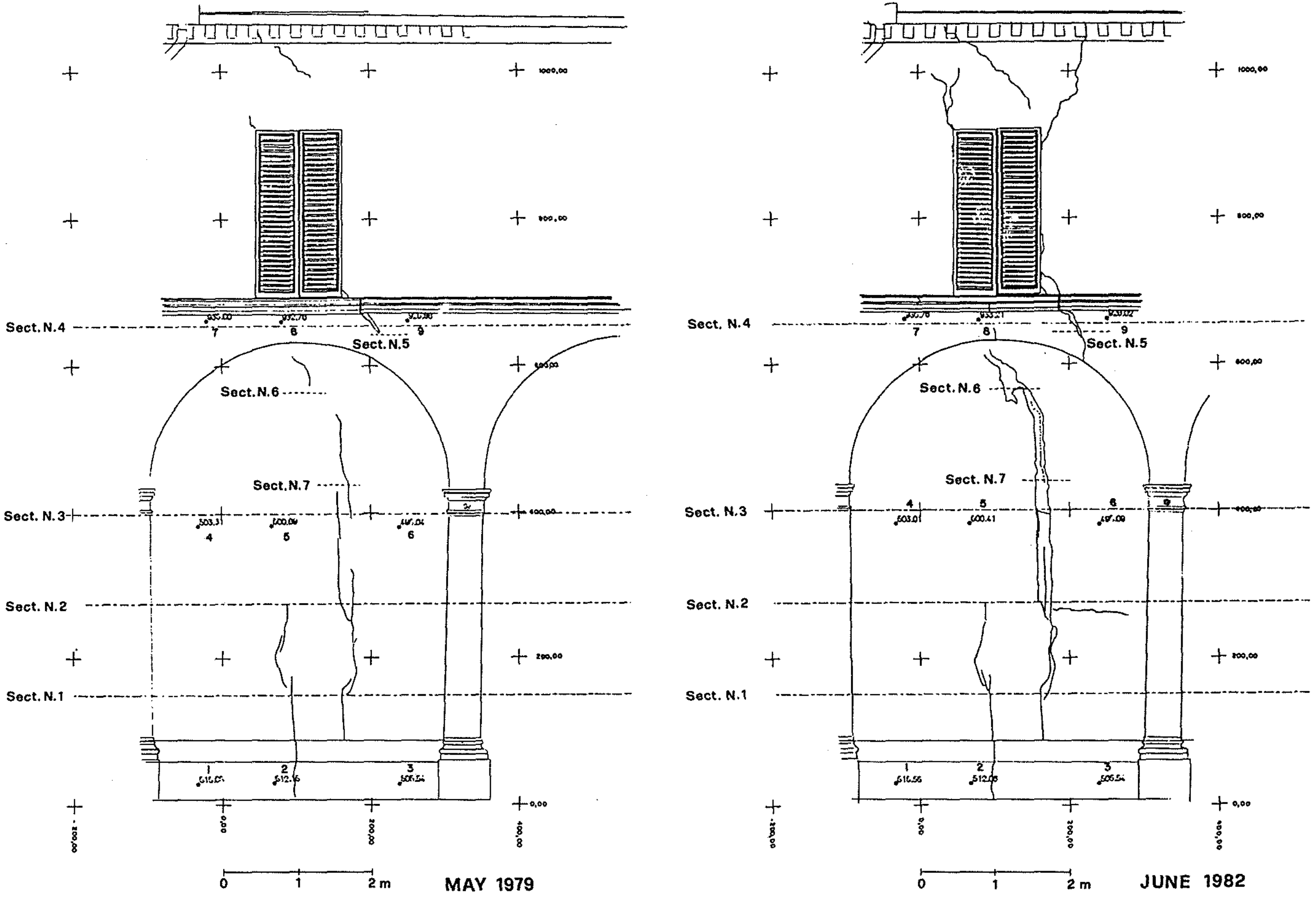

FIG. 6. Prospects relative to the first and last survey.

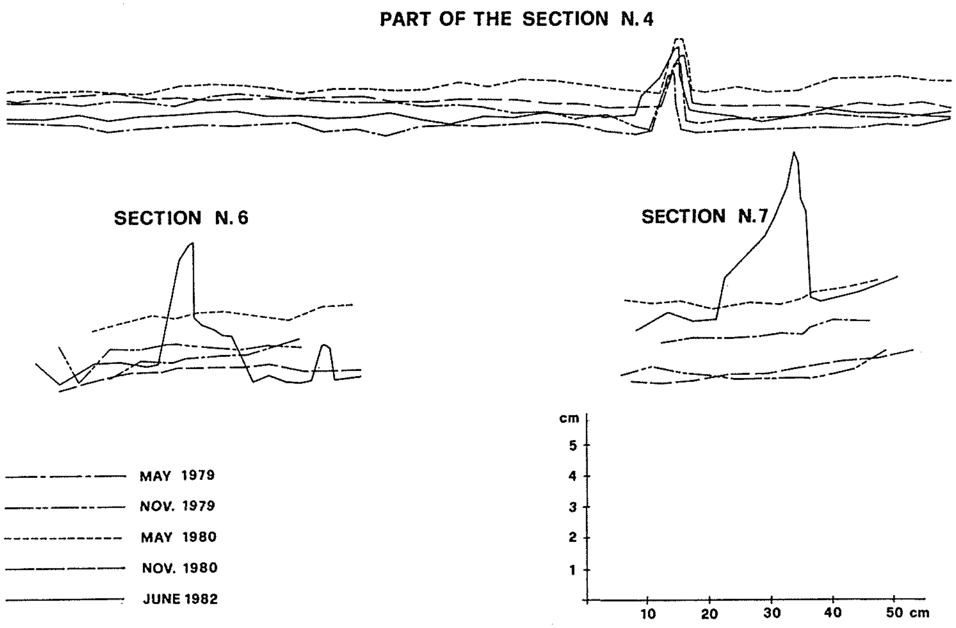

FIG. 7. Horizontal sections of the wall.

wall, cameras have been set up on the same vertical line, at a distance d of about 6 m; being the distance D between cameras and wall about 18 m, the rate d/D has resulted about 1/3.

The orientation points have been materialized by plunging into the wall 9 metal little cylinders with a cross engraved on an extreme for theodolite collimation; the position of these points have been determined with reference to a permanent quadrilateral set up in the cloister court. Angles and distances regarding such network have been measured at every survey; height differences between points have been determined by high precision levelling. The variations of the co-ordinates of points occurred between measurement campaigns do not exceed measurement errors; therefore we have considered the quadrilateral as a stable network, from which we have surveyed the orientation points on the wall at every campaign.

After having determined the positions of orientation points, the plotting of photogrammetric models has been performed by a stereocomparator. The progressive damage of the wall can be noticed by the succession of the prospects relative to the five surveys; in FIG. 6 only the first and last one are reported for the sake of brevity. Moreover seven horizontal sections of the wall have been defined in different positions; in FIG. 7 we have reported only the most interesting ones. Such sections give a metric indication of damage progression.

Acknowledgment

The authors have to thank Marco Del Duca and Gaetano Scuderi for their cooperation in some measurements.

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SIMULATION OF FLOW AND COMPACTION IN THE REGIONAL AQUIFER SYSTEM OF THE CENTRAL VALLEY OF CALIFORNIA, U.S.A.

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Abstract

The largest volume of land subsidence due to ground-water pumpage in the world, 21,000 hm^3 , has occurred in the Central Valley. This has resulted from a combination of the largest average annual ground-water pumpage per unit area, 0.27 m, for any regional aquifer system in the United States and probably the world, and a large aggregate thickness of compressible clay beds in the aquifer system. Although land subsidence was a very significant result of ground-water pumpage at some locations during some time periods, its volume is relatively small compared to either the volume of ground water in storage or the volume pumped annually in the Central Valley. Vertical ground-water flow is at least as significant as either horizontal flow or inelastic compaction in the Central Valley aquifer system. Simulation of ground-water flow and aquifer compaction using a three-dimensional finite-difference computer model resulted in a change in our concept of the aquifer system.

Introduction

A regional analysis of the Central Valley aquifer system (fig. 1) was done as a part of the nationwide Regional Aquifer System Analysis (RASA) program of the U.S. Geological Survey (Bertoldi, 1979, and A. K. Williamson and others, in prep.). The principal method of analysis was a digital flow model incorporating inelastic compaction of the aquifer system. The purpose of this paper is to describe the principal results of the analysis that relate to land subsidence and to compare volumes of water released from inelastic compaction with other sources of water from the regional aquifer system. Many data relating land subsidence to hydraulic-head declines have been collected. A flow model allows investigation of the relation of land subsidence to pumpage and physical and hydrologic properties. An understanding of the interrelations between pumpage, recharge, ground-water flow, water-level changes, and land subsidence is necessary to determine the effects of ground-water pumpage on the aquifer system. The methods developed in this study will also be useful to another RASA study now being conducted of the Gulf Coastal Plain where land subsidence due to ground-water pumping also is a problem (Gabrysch, 1982).

The Central Valley of California is a large (52,000 km^2) structural trough of unconsolidated sediments, about 650 km long and 80 km wide. The mean annual precipitation on the valley floor ranges from 650 mm at the north end to 130 mm at the south end of the valley. Almost all of the precipitation occurs during the winter. Irrigated agriculture began more than 100 years ago and is practiced on more than 57 percent of the land area. About one-half of the 27,000 hm^3 of water used annually for irrigation is pumped from the aquifer system (A. K. Williamson and others, in prep.). This volume of pumpage is equivalent to a depth of about 0.27 m

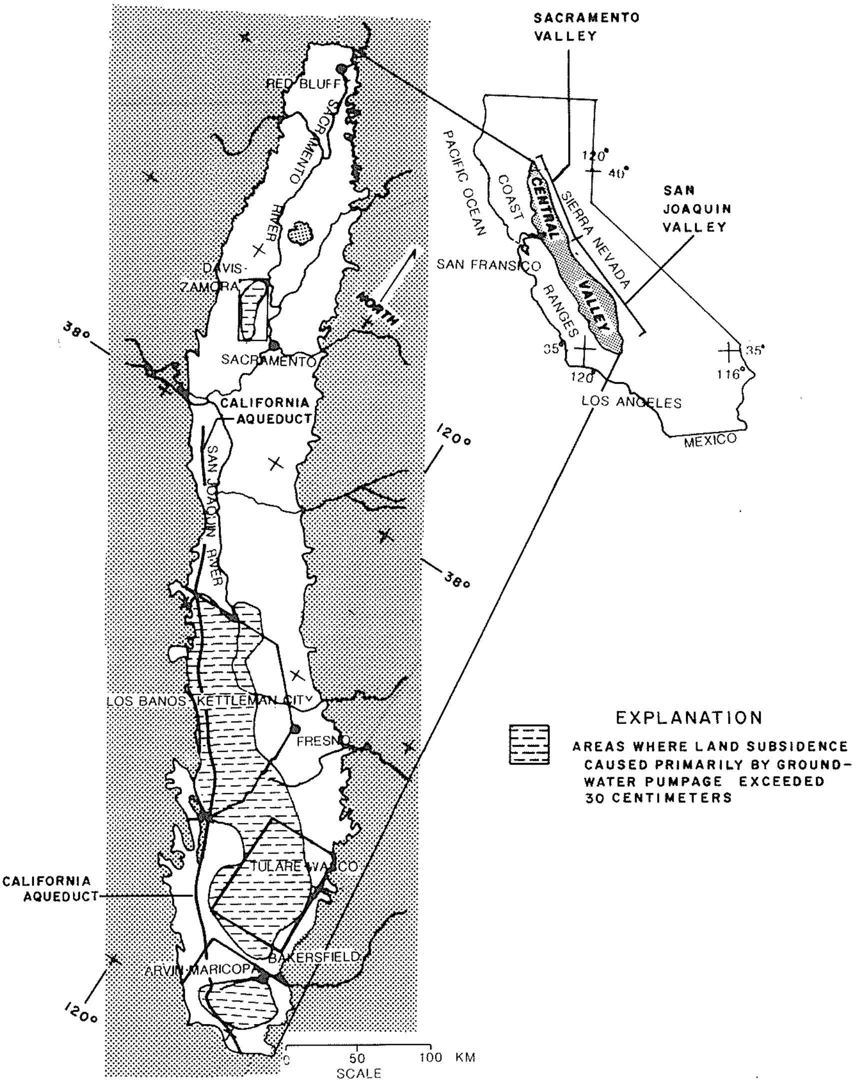


Figure 1.-Location of the Central Valley of California and the principal areas of land subsidence.

over the entire valley floor. Twenty percent of the annual ground-water pumpage in the United States occurs in the Central Valley. Ground water is important as a stable source of supply because of the extreme variability of surface water from winter to summer and from year to year. The volume of fresh ground water in storage is more than 25 times the storage capacity of surface reservoirs in the valley. Therefore, ground water is necessary as an alternative supply for surface-water users. This was demonstrated during the 1976-77 drought, when surface storage was depleted and many farmers switched to ground water for irrigation. About 80 percent of the pumpage in the valley occurs in the San Joaquin Valley, where most of the land subsidence has occurred. Hydraulic-head declines of as much as 120 m have resulted from the ground-water development. The largest volume of land subsidence in the world due to ground-water pumpage, 21,000 hm^3, has occurred in the Central Valley (J. F. Poland, oral comm., 1982). More than 13,500 km^2 of the San Joaquin Valley have been affected by subsidence and at one location, subsidence has exceeded 9 m (Ireland and others, 1982).

The model

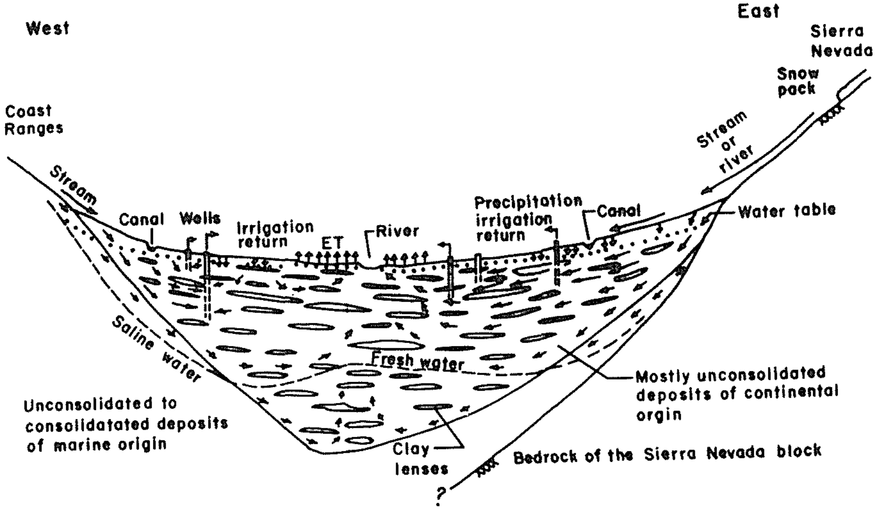
Ground-water flow and aquifer compaction were simulated using a three-dimensional finite-difference computer model. The computer program that was used in the analysis was written by Trescott (1975), and modified by Trescott and Larson (1976) and Torak (1982). In order to account for the release of water from inelastic (irrecoverable) compaction of fine-grained beds in the aquifer system, the program was modified in a manner similar to the method used by Meyer and Carr (1979). To briefly describe the modification: The program includes both an elastic and an inelastic storage coefficient, using the inelastic value only when the hydraulic head in the aquifer at the end of the last time step was equal to or lower than its previous lowest value. A companion paper discusses, in detail, the method used in the model (Prudic and Williamson, 1984).

The model used a square grid with a horizontal spacing of 10 km due to the regional nature of the study. Most of the calibration and testing of the model was done using 1961 to 1977 data. Pumping periods of 6 months were used to simulate the large seasonal fluctuations that occur because of the dominant summer agricultural pumpage and winter recharge.

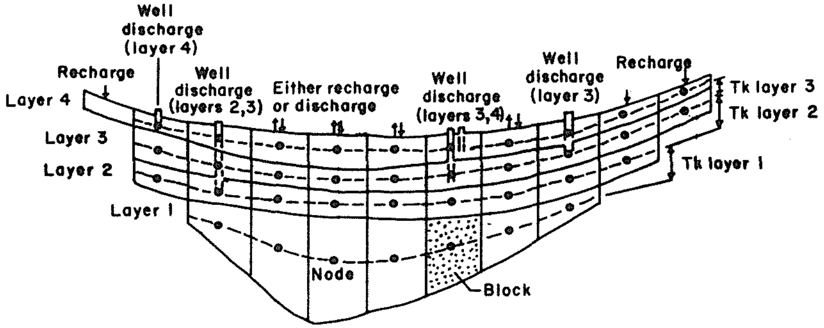
Concept of the aquifer system

The regional flow analysis has resulted in a change in our concept of the aquifer system. The aquifer system is composed of equal thicknesses of intercalated coarse- and fine-grained continental sediment; with total thickness ranging from about 600 m at the north end to about 2700 m at the south end of the valley. Most of the beds are areally discontinuous, although some, like the Corcoran Clay Member of the Tulare Formation, extend throughout one-third of the valley. Most investigators have conceptualized the northern one-third of the valley, the Sacramento Valley, as one water-table aquifer and the southern two-thirds, the San Joaquin Valley, as two aquifers generally separated by a regional confining layer, the Corcoran Clay Member. Our simulations show that the large number of fine-grained beds restrict vertical flow, even in areas and at depths where no regional confining layer exists. Our concept of the aquifer system is that the entire thickness of continental deposits is one aquifer system with varying vertical hydraulic conductivity and consequently, varying degrees of confinement, depending on the proportion of fine-grained sediments and other factors (fig. 2). The thickness of the aquifer system was divided, not necessarily at geologic contacts, into a maximum of four model layers for simulation. The top model layer was simulated as having water-table conditions. The lowest model layer was below all of the pumped zones. Wells are perforated in permeable zones corresponding to one or two adjacent layers of the top three layers. The vertical permeability was

A. Aquifer system



B. Model of the aquifer system



Arrows indicate general direction of water movement. Longer arrows imply larger flows.

Tk layers are calculated by dividing the harmonic mean of the vertical hydraulic conductivity of the aquifer materials by the thickness between nodes. Tk layers 2 and 3 may be affected by wells that are screened in both layers 2 and 3 and less frequently in layers 3 and 4.

Discharge from wells is simulated at the node.

Note: Sketches are not drawn to scale.

Figure 2.-Diagramatic cross-section and concept of the aquifer system

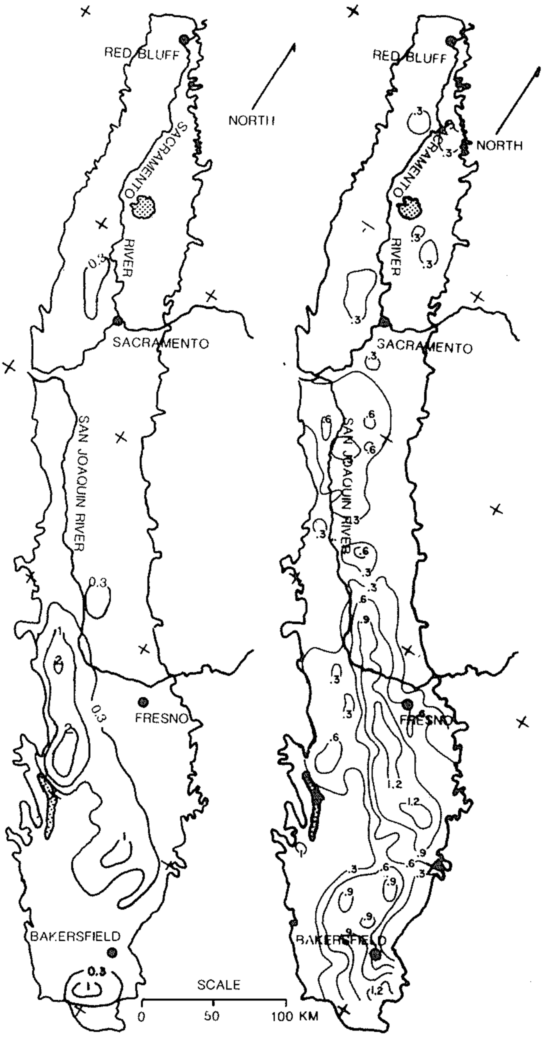
allowed to increase from its predevelopment value where a significant number of individual well perforations spanned two adjacent layers.

Significance of inelastic compaction and subsidence to ground-water flow
Although land subsidence was a very significant result of ground-water pumpage at some locations during some time periods, its volume is rela-

tively small compared to either the volume of ground water in storage or the volume pumped annually in the Central Valley. About 1 million hm^3 of fresh ground water was in storage in the upper 300 m of the aquifer system before development began. This volume of ground water in storage has decreased by about 74,000 hm^3 from predevelopment conditions until 1977. Of this decrease, about 49,000 hm^3 (67 percent) was derived from the lowering of the water table, 21,000 hm^3 (28 percent) from inelastic compaction, and 4,000 hm^3 (5 percent) from elastic storage. During 1961-77, ground water withdrawn from storage averaged about 1,000 hm^3 per year. This was only 7 percent of the pumpage (although locally it was as much as 60 percent), with the remaining pumpage being supplied from recharge. The recharge to ground water had increased more than one order of magnitude due to development, especially through percolation of irrigation water.

Generally, studies of land subsidence due to ground-water pumpage have focused on the relation of land subsidence to water-level decline. Consequently, in some areas, when subsidence has exceeded acceptable amounts, water managers have tried to eliminate pumpage from the area entirely to reverse the trend of declining water levels. A more useful relation for water management would be the relation between land subsidence and pumpage, such as can be obtained from a ground-water flow model. This is not a simple relation as indicated by comparing the distribution of 1961-75 land subsidence (resulting from inelastic and elastic compaction) and ground-water pumpage for a typical year, 1961 (fig. 3). There are other factors that affect this relation such as the depth interval at which the wells are perforated and the composition of the aquifer materials. Pumping from shallower depths generally minimizes the volume of subsidence because (Prudic and Williamson, 1984): (1) A greater proportion of the pumpage comes from the water-table zone where less hydraulic-head decline occurs for a given volume of pumpage because water released from storage comes mostly from dewatering of aquifer materials, which is greater than that released from elastic storage, and because recharge is induced by the declining water table; and (2) this decreases the downward vertical hydraulic-head gradient between the water table and the lower pumped zone, which decreases the seepage stress. The thickness, compressibility, and mineralogy of the fine-grained deposits also affect the volume of subsidence that will occur in a specific area from a specific volume of pumpage (Prudic and Williamson, 1984).

The change in the annual volume of recharge from pre- to postdevelopment conditions is nearly as much as the volume of land subsidence which occurred from 1930 to 1977. The total recharge and total discharge have both increased more than 40 times compared to predevelopment values due to ground-water development. The types and distribution of recharge and discharge also have changed due to development. During 1961-77, recharge was derived mostly from irrigation return flow (81 percent), but also from precipitation (14 percent) and streams (5 percent). The volume from irrigation return flow is somewhat surprising because soil characteristics were thought to impede percolation of irrigation water. The actual proportion of recharge from streams is probably larger than 5 percent, but due to the large size of the regional model blocks, some stream recharge is discharged to other stream reaches within the same model block and does not appear in the water budget simulated by the model. Average annual surface-water outflow from the valley has decreased from about 30,000 hm^3 to about 19,000 hm^3 mainly due to increased evapotranspiration from irrigated cropland.



A. 1961 – 75
Land subsidence,
in meters

B. 1961 Pumpage
per unit area,
in meters

CONTOUR INTERVAL VARIABLE

Figure 3.-Land subsidence, 1961-75 and ground-water pumpage for a representive year, 1961.

<u>Significance of vertical flow in the aquifer system</u>

Vertical ground-water flow is at least as significant as either horizontal flow or inelastic compaction in the Central Valley aquifer system. In addition, the vertical patterns of well perforations seem to have more effect on vertical ground-water flow than the distribution of fine- and coarse-grained sediments. Water levels in wells in many areas of the valley have declined significantly due to ground-water pumpage. The declines have been much larger in wells completed in the deeper pumped zones because the storage coefficient is smaller than the specific yield of the shallow zone. There has been no dewatering of aquifer materials in the deeper zone. The decline, of hydraulic head, has greatly changed the direction and magnitudes of both horizontal and vertical flow. In general, vertical flows in model blocks were larger than horizontal flows even though vertical hydraulic conductivities were several orders of magnitude smaller than horizontal hydraulic conductivities. This is because vertical hydraulic gradients are much larger than horizontal hydraulic gradients due to the shorter distances involved. Flow area also is much greater in the vertical than it is in the horizontal. These two reasons are not only related to the geometry of the model blocks but also to the overall geometry of the aquifer system. Depths generally are less than 1 km whereas while horizontal dimensions are several tens of kilometers.

Vertical flow is very significant in the Los Banos-Kettleman City area, where the greatest volume of land subsidence in the valley has occurred. Prior to development, the vertical flow in this area was mostly upward toward natural discharge areas. Pumpage reversed that gradient and by the early 1960's, the difference in hydraulic head between the lower pumped zone and the water table was more than 120 m in several parts of this area. After 1968, the hydraulic-head difference between the two zones decreased due to decreased pumping when more surface water was delivered to the area through the California Aqueduct. The simulated flow of water to and within the Los Banos-Kettleman City area during the early 1960's when the rate of land subsidence was near its peak is shown in figure 4. Downward leakage from the shallow ground water accounted for about 32 percent of the pumpage from the lower pumped zone. Despite large horizontal hydraulic-head gradients from east to west during this period (as much as 50 m per 10 km), only about 13 percent came from ground-water flow from the east side of the valley. Inelastic compaction was the source for 47 percent of the lower-zone pumpage, and the remaining 8 percent came from elastic storage and upward leakage from below the lower pumped zone. Downward leakage from the water table is an important component that affects the volume of subsidence that occurs because if one component is restricted, the others will have to increase to compensate.

Long perforated intervals of many wells have a great effect on the vertical resistance to flow by allowing a route for leakage between different zones in the aquifer system. Most of the more than 100,000 irrigation and public supply wells in the valley are perforated throughout the entire lower two-thirds of their depth. Davis and others (1964, p. 81-88) discussed the "interaquifer circulation of ground water" across the Corcoran Clay Member through well casings in the Los Banos-Kettleman City area. More data are now available and it seems that most of the vertical flow measured in those wells with current meters was actually vertical flow within the lower pumped zone responding to differential pumping stresses. Under large vertical hydraulic-head differences, a few wells with long perforated intervals can transmit as much water as that which leaks through the sediments within an entire model block. This can be shown by applying the Thiem equation, assuming that the hydraulic-head loss for water flowing down the wells occurs in the aquifer, flowing to and from the well. If the more than 30 wells per block were perforated across the boundary between the zones, enough water would flow down the wells to dissipate the large actual vertical hydraulic-head differences (A. K. Williamson, and others,

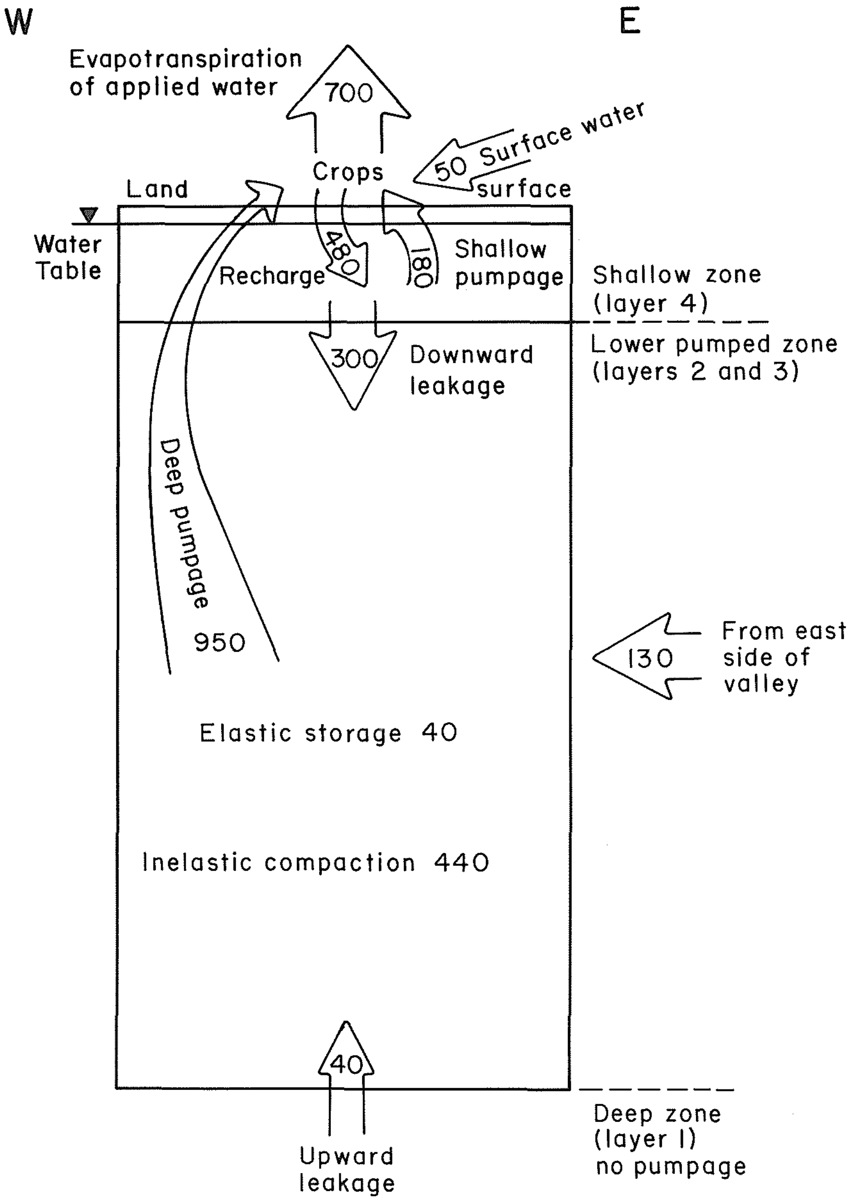


Figure 4.-Sources and flows of water in the Los Banos-Kettleman City area in the early 1960's (hm^3/yr).

in prep.). Therefore, the perforations in most of the wells in the Los Banos-Kettleman City area must not connect the shallow water table unit with the lower pumped zone. However, the leakage of water in wells with long perforated intervals within the lower pumped zone effectively equalizes pressures throughout the zone. There were many places where the water level in a well 200 m deep was nearly equal to the one in a well 700 m deep nearby, though they were both much different from the water level in a 50 m well. This situation required, wherever possible, the division of the aquifer system into layers at zones that were not crossed by the perforated intervals of wells in order to adequately simulate the actual hydraulic-head gradients.

Implications for future development

The new concept of the aquifer system developed by analysis of the flow system could be used in ground-water management in the Central Valley. It should help define land subsidence which could occur in areas that have not been substantially affected yet.

Ground-water pumpage could be managed so that subsidence could be minimized. Although the volume of inelastic compaction and resulting land subsidence has been substantial, it has still been relatively small (about 2 percent) compared to the total volume of ground-water pumpage for the whole valley during 1961-77. Despite large volumes of ground-water pumpage during the last 100 years, only about 7 percent of the approximately 1 million hm^3 of fresh ground water that is in storage in the upper 300 m of the aquifer system has been withdrawn. During 1961-77, the average annual volume of withdrawal from storage was less than 1 percent of the ground water in the upper 300 m of the aquifer system. Due to increased surface water imported, this percentage is now even less.

Ground-water development and the resultant water-level declines has caused natural discharge to be captured and an increase in both natural and artificial recharge to the aquifer system. Average annual surface-water outflow from the valley has decreased about 12,000 hm^3 since water development began. If there were no further increases in pumpage, the aquifer system would eventually reach an equilibrium where discharge was matched by recharge and no further water-level declines would occur. This would have already occurred, except that as pumpage has decreased in some areas as more surface water was imported, pumpage has greatly increased in other areas where new land is being brought into production. Therefore, the aquifer system has not reached equilibrium before a new increase began. Land subsidence is increasing in some areas (Lofgren and Ireland, 1973) and the potential exists for subsidence in other parts of the valley as well.

The volume of land subsidence in a specific area that occurs from a given volume of pumpage from deep zones is not only related to the composition of the aquifer materials, but also the pattern of pumpage, vertically and horizontally. For a desired volume of ground-water pumpage in the valley, wells could be located areally and with depth to get the most pumpage with the least land subsidence. This, however, would require a change in legal and governmental restraints on ground-water management, as well as consideration of other factors such as water quality.

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CLASSIFICATION OF LAND SUBSIDENCE BY ORIGIN

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Abstract

Data collected during the last decades allow classification of subsidence into 2 groups: Endogenic, caused by processes originating within the planet; and Exogenic, caused by processes originating near the Earth's surface. Exogenic subsidence can be subdivided into subsidence caused by removal or weakening of support and subsidence caused by an increase in actual or effective loading. Because of interrelationships between natural processes, this classification is only an approximation; subsidence is frequently related to compaction at great depth. Preservation of porosity here can be attributed to the tectonically controlled piezometric head developed prior to or during deposition of overburden. No subsidence will take place as a result of decline of piezometric head developed after aquifers became compacted by overlying sediments.

This concept allows detection of aquifers susceptible to subsidence by consolidation testing; samples of aquifers susceptible to subsidence are capable of consolidation at loadings less than the weight of their overburden.

Introduction

Since the middle of the 20th century, certain forms of land subsidence, related to human activity, have become a widespread, well-recognized geologic hazard (for example, Bolt, et al., 1975). The process has caused large monetary losses (Prokopovich and Marriott, 1983) and, not infrequently, human losses. The future spread of subsidence with growing world population and increasing technological advancement should be anticipated (Prokopovich, 1972). Methodologically, the first two steps in any scientific study are: (1) the accumulation of factual data and (2) proper classification of these data. Several national and international symposia and conferences on land subsidence have occurred during recent decades and a large number of factual case histories of subsidence have accumulated worldwide (Anonymous 1969, 1977; Donaldson, 1983; Poland, in print; Saxena, 1979; etc.).

This information provides a good background for the second step - the classification of subsidence. Such classification can be based on several principles, such as monetary impact of subsidence, magnitude of subsidence, geographic spread of subsidence, etc. From a geologic standpoint, however, the most desirable and scientifically correct classification should be based on the origin of the process, i.e., on its genesis.

The following genetic classification of land subsidence (Fig. 1) was originally proposed at the 24th International Geologic Congress in Montreal, Canada (Prokopovich, 1972), and repeated with minor modification at the International Conference on Evaluation and Prediction of Subsidence (Prokopovich, 1979). In order to promote this classification, regardless of previous presentations, its discussion at the present international gathering seems to be appropriate.

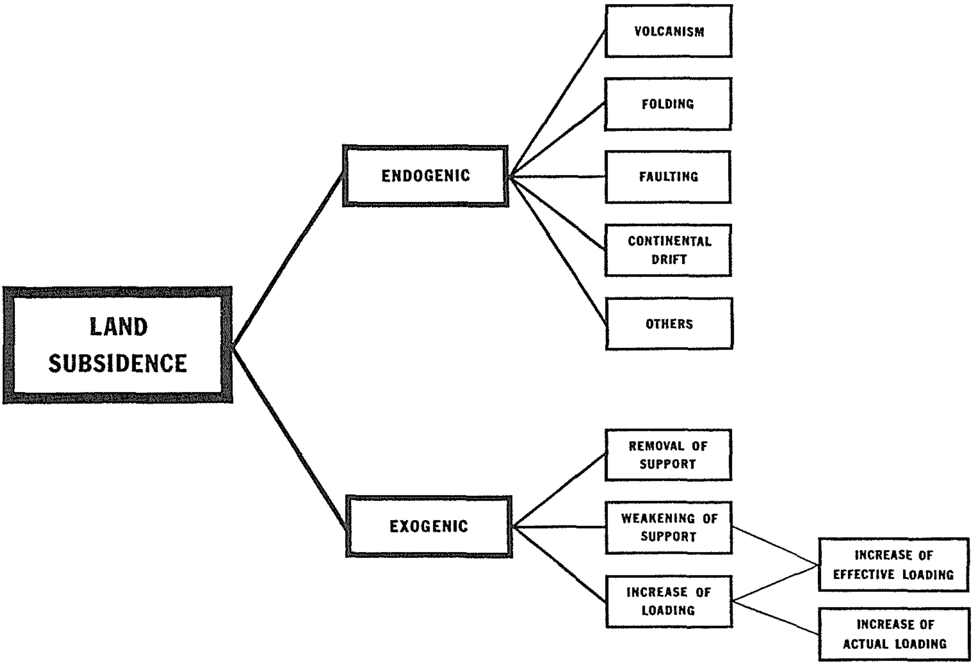

Fig. 1. Genetic classification of land subsidence.

Subsidence and Its Spread

The term "subsidence" is as old as "modern geology." It is generally accepted that geology was "born" with the publication of Charles Lyell's Principles of Geology or the Modern Changes of the Earth and Its Inhabitants in the middle 1800's. The word subsidence appears on the first page of this classic volume in the caption to the frontispiece picture of ruins of Serapis' temple in Pozzuoli, Italy. The word was also used in a similar way in many places in the text of the book to indicate, mostly, the results of sea transgressions, rather than actual lowering of land surface. According to the modern definition by Bates and Jackson (1980, p. 624), subsidence is the "sudden sinking or gradual downward settling of the Earth's surface with little or no horizontal motion...."

Such "land subsidence" was rarely mentioned in the geologic literature 50-60 years ago, but has become alarmingly common, particularly after World War II. At the present time, exogenic subsidence has been reported on all continents, in different climatic zones, and is recognized as both a major geological hazard (Bolt, et al., 1975), and a form of pollution (Prokopovich, 1978). What are the reasons for such a "renaissance"? Was land subsidence simply not recognized in the past? Perhaps this is true in some cases, but in general, such an explanation is not sufficient.

Why did our dependable "terra firma" become "terra infirma"? Actually, the recent rapid spread of exogenic subsidence caused by human activity simply reflects the unprecedented growth of world population, growth of affluent societies, and the advancement of technololgy.

Genetic Classification

According to the classification, all types of subsidence can be divided into two major groups - ENDOGENIC SUBSIDENCE, caused by processes originating within the planet, and EXOGENIC SUBSIDENCE, resulting from forces originating on the Earth's surface and being ultimately related to solar energy. This second group also includes the consequences of ever increasing, frequently poorly planned, human activity. Each of the major groups can be subdivided according to the nature of processes causing the subsidence. The following text contains random, illustrated examples of such divisions and subdivisions.

Classification of endogenic subsidence is rather simple. It can be subdivided into subsidence caused by volcanic activity, folding, faulting, and other endogenic processes.

An excellent example of subsidence caused by volcanic activity is Crater Lake located in Oregon, U.S.A (Fig. 2A). It is generally believed that large volcanic eruptions of the ancient Mount Mazama partially emptied the magma chamber feeding this giant volcano, resulting both in the collapsing of its top and creation of Crater Lake. Another probably more common, but less spectacular type of volcanic subsidence is the collapsing of roofs of lava caves.

A good example of subsidence caused by folding is the Central Valley of California (Fig. 2B). This valley is a giant structural trough, located mostly between two mountain ranges - the Sierra Nevada on the east, and the Coast Ranges on the west. The folding here is probably related to plate tectonics.

Typical cases of subsidence related to faulting are numerous grabens or down-faulted blocks of the Earth's crust, such as the famous Rhine Valley in Europe. In the United States, "horst and graben" topography is particularly well developed in the Basin and Range Province in Nevada, Arizona, and partially, in California (Fig. 2C).

By definition, all forms of endogenic subsidence are natural processes not related to human activity. On the contrary, many forms of exogenic subsidence are directly caused by human activity. The impact of this subsidence will increase with increasing world population and advancing technology. This subsidence is actually a surficial expression of compaction ("consolidation") of material at depth due to removal of support (for example, oxidation of organic particles, melting of ice, etc.), weakening of support (for example, hydrocompaction), or an increase in actual or effective loading. Good examples of subsidence due to an increase in actual loading are collapses of karst cavities due to dewatering of unconfined aquifers. Such dewatering results in both increased grain-to-grain pressure due to the loss of buoyancy and increased weight of water-coated particles (Poland and Davis, 1969). Typical examples of subsidence due to an increase in effective loading are subsidence related to piezometric decline of confined aquifer systems and subsidence due to withdrawal of crude oil and natural gas. Selected examples of these forms of subsidence are shown in Fig. 3.

Interaction of Exogenic and Endogenic Processes

The presently accepted definition of land subsidence and its genetic classification should be considered as only a "first approximation" and may raise several new, hard-to-answer questions. For example, determination of vertical displacement of a point is usually made by comparing its surveyed elevations. "Absolute" elevations are based on mean sea level, which is not a constant. During Pleistocene glaciation, much ocean water

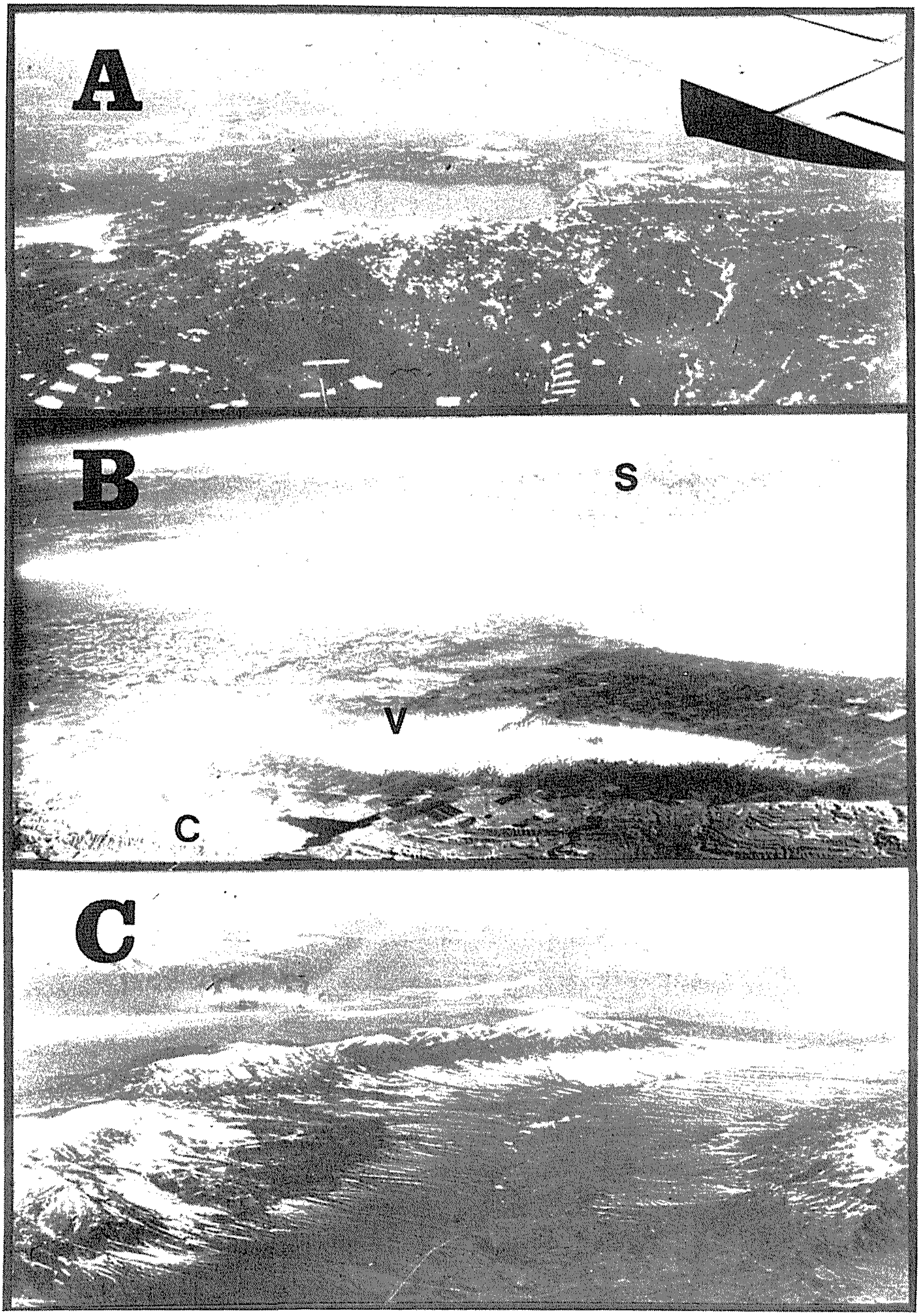

Fig 2. Selected examples of endogenic subsidence due to (A) volcanism (Crater Lake collapse caldera in Oregon, U.S.A.); (B) folding (Central Valley (V), located between the Coast Ranges (C) and the Sierra Nevada (S), California, U.S.A.); and (C) faulting (horst and graben topography - Basin and Range province, Nevada, U.S.A.).

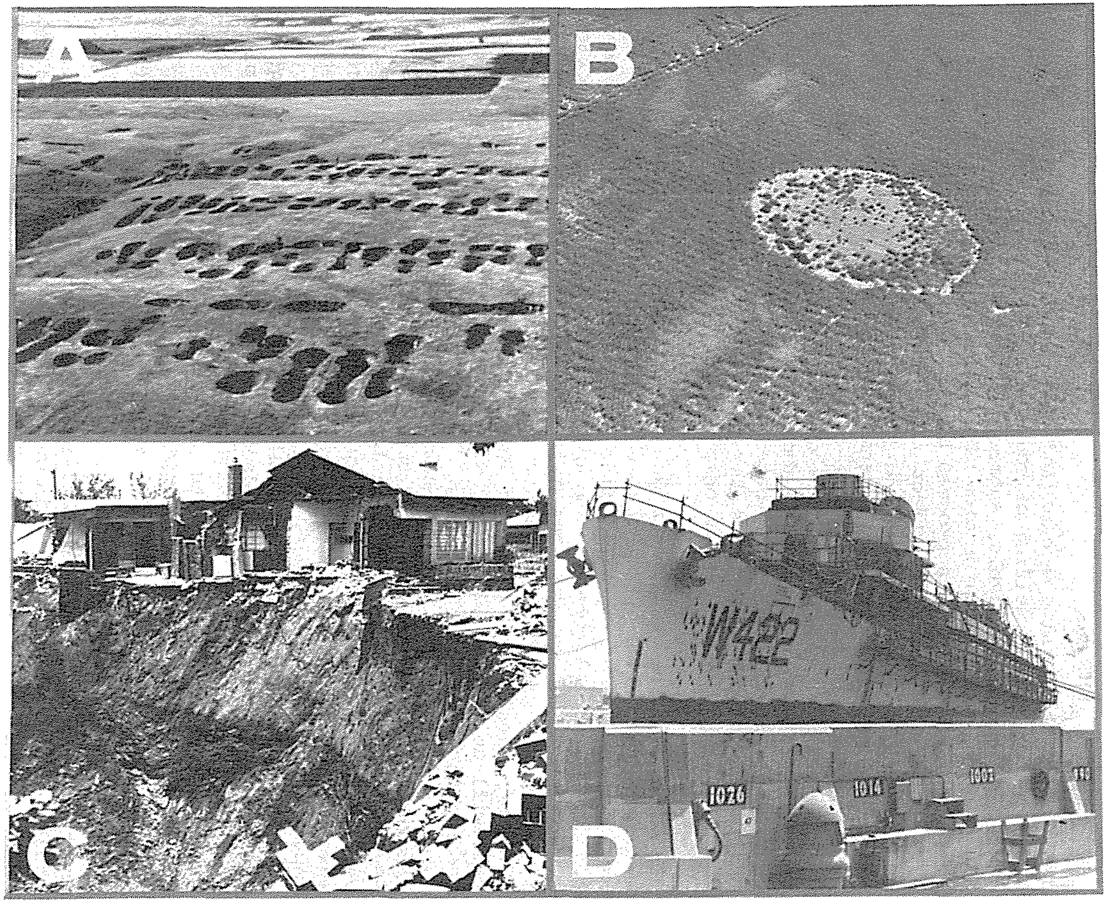

Fig. 3. Examples of exogenic subsidence due to (A) removal of support (collapsed underground lignite mines, North Dakota, U.S.A., courtesy of the U.M. Region of the USBR); (B) weakening of support (hydrocompaction sink in the San Joaquin Valley, California, U.S.A.); (C) increase of actual loading (collapse of preexisting karst cavity due to dewatering for gold mining - loss of buoyancy and increased grain to grain stress, South Africa, courtesy of Dr. C. A. Bezuidenhout); and (D) increase of effective loading (withdrawal of crude oil; dry dock in Long Beach Harbor, California, U.S.A., courtesy of U.S. Navy).

was bound in continental ice caps and global sea levels were well below their present position. Melting of continental ice released large volumes of water and raised sea levels, submerging near-shore lowlands. If such flooding is considered to be a form of exogenic subsidence, we can expect a global acceleration of this process because of the melting of polar ice due to a growing "greenhouse" effect resulting from increasing levels of carbon dioxide and other pollutants in the Earth's atmosphere.

A more basic complication of genetic classification is the existence of intimate interrelationships between natural processes. As a general rule, exogenic processes are controlled by an endogenetic framework. For example, studies of subsidence caused by overdraft of confined aquifer systems in California indicate the important function of tectonic movement in this process. Since the subject is discussed in detail by Prokopovich (1983, 1983A), it is only briefly outlined in the following text.

Subsidence in the California San Joaquin Valley is mostly caused by an overdraft of a deeply seated sub-Corcoran aquifer system, and is a surface expression of consolidation of sediments of this aquifer at depth. In both the laboratory and nature, a series of equal increments of increased loading in the same material will result in progressively less compaction. Graphically, such a relationship follows an exponential decay curve. Hence, at a certain loading the original porosity of the material will be reduced to a point from which no notable further compaction will take place. Two terms, "stable depth" and "stable field," were used by Prokopovich (1976) to express these concepts. Depending upon the composition of sediments and other geologic factors, the "stable depth" below which additional compaction of sediments is practically nil, may vary. Theoretically, the amount of potential subsidence exponentially decreases with depth, and near-surface deposits are the most susceptible to compaction-related subsidence.

Surprisingly, most of the available facts on compaction of aquifers in areas affected by major subsidence contradict this assumption and indicate that most compaction actually occurs at a relatively great depth. The contradiction can be explained by past tectonic movements, which created piezometric head, which compensated for some of the loading of newly deposited overburden and thus prevented "normal" consolidation of aquifer systems (Fig. 4).

Prediction of Susceptibility of Aquifers to Subsidence

The theoretical considerations described above may be used for relatively simple and inexpensive detection of aquifer systems susceptible and not susceptible to compaction due to an increase in effective loading, i.e., systems capable and not capable of causing land subsidence.

Such detection can be achieved by one-dimensional consolidation testing of undisturbed core from the studied sediments. Routinely, consolidation testing is conducted on samples initially prestressed by loading equal to the weight of overlying sediments. For detection purposes, however, the initial testing should start with loading equal to approximately one-fourth to one-third of the weight of the overlying sedimentary column. The following testing should be carried out with increasing load increments, approaching, reaching, and exceeding the weight of overburden.

Interpretation of changes in the trend of consolidation-loading graphs will theoretically indicate the susceptibility of tested samples to consolidation under an increase in effective loading, i.e., susceptibility of the system to subside due to a decline in piezometric pressure. The

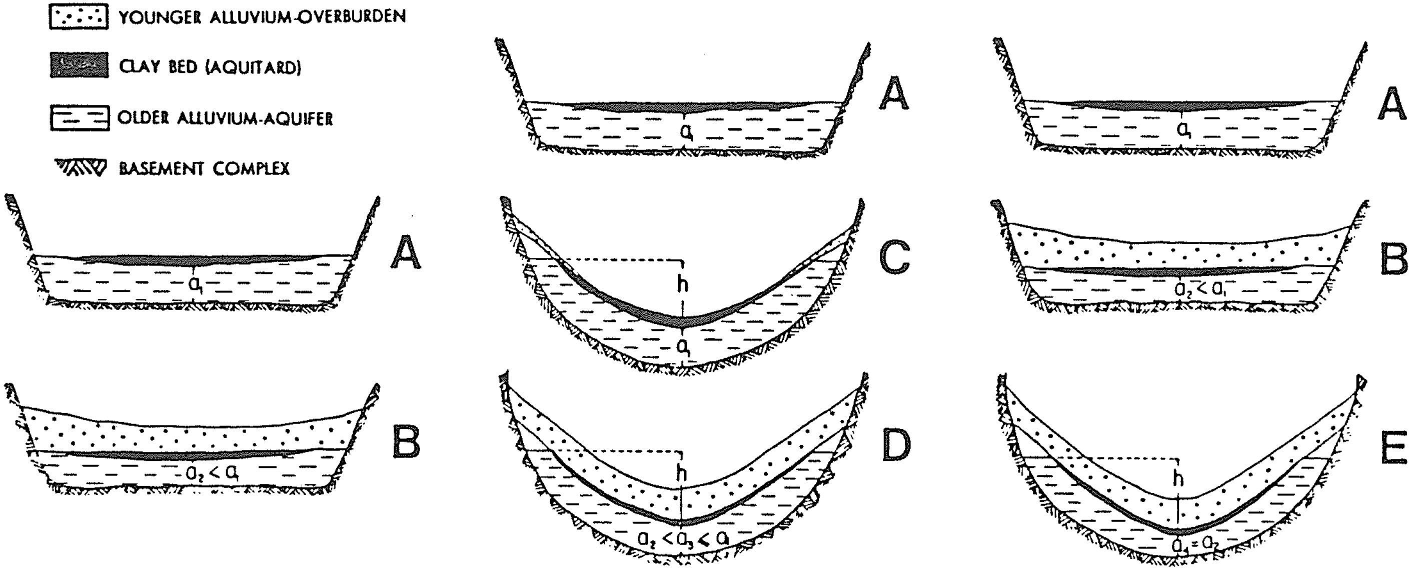

Fig. 4. Theoretical relationship between susceptibility to compaction and piezometric head of an aquifer. A. Older alluvium aquifer capped with a clay bed in an intermountain basin before deposition of overburden. B. The aquifer is compacted by the weight of overburden. C. Folding creates a confined aquifer in older alluvium prior to or during the deposition of overburden. D. Piezometric head partially compensates for the weight of overburden deposited after or during folding. Compaction of the aquifer is less than in case B. Decline of piezometric head under such conditions will result in subsidence. E. Folding occurred after the deposition of overburden; the aquifer is compacted by weight of overburden. No subsidence will be caused by a decline of piezometric head under such conditions.

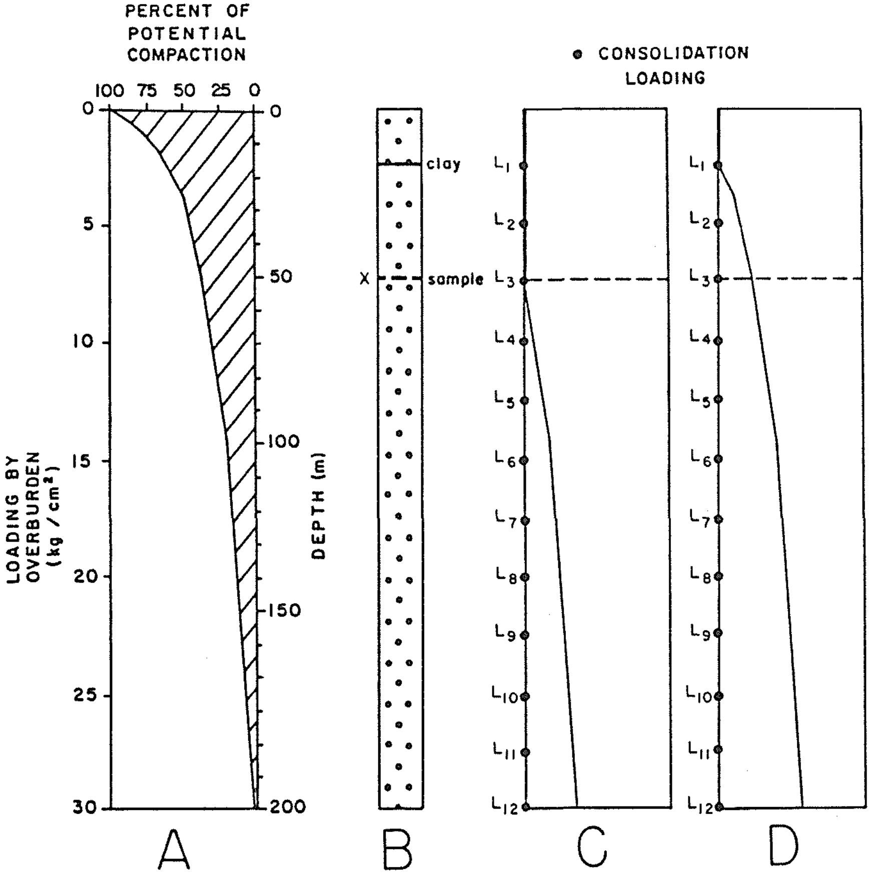

Fig. 5. Theoretical prediction of susceptibility to subsidence. A: An example of a theoretical graph showing a decrease in porosity-compaction with increasing depth in a more or less homogeneous column of sediments B. C: Theoretical laboratory consolidation graph of an undisturbed soil sample "X" taken from a "normally compacted" soil column. No notable compaction occurred at loadings which are less than or equal to the weight of overburden. D: Theoretical laboratory graph of an undisturbed soil sample "X" taken from an aquifer susceptible to subsidence. Notable consolidation starts at loadings which are less than the weight of overburden.

idealized interpretation of compaction graphs is illustrated in Fig. 5. In the figure, compaction is shown not as the usual semilogarithmic plot, but as a Cartesian compaction-loading graph (Fig. 5, graph A). The loading scale is modified to indicate both the loading and the depth in an assumed basin. Graph A illustrates a "normally compacted" (consolidated) condition not modified by piezometric pressure.

A theoretical graph of laboratory consolidation of a sample taken at point "X" is shown in Fig. 5, graph C. Due to natural loading, the first three loadings (L_1, L_2, and L_3) of the sample will cause no consolidation. (Actually, minor consolidation due to rebound will occur during this initial loading.) However, consolidation will occur at the loadings L_4, L_5, L_6, etc., which exceed the weight of material capping sample "X". The confined aquifer system described in this testing was completely "precompacted" by the load of overburden, and the terrain is not susceptible to subsidence due to decline in piezometric head.

In the case shown in Fig 5, graph D, notable laboratory consolidation of the sample started at the loading L_2, which is smaller than the calculated load of overburden at the depth of the sample. Such compaction (consolidation) patterns indicate deposits that are not completely preconsolidated by the weight of overburden and, hence, are susceptible to compaction as a result of an increase in effective loading. Land subsidence due to a decline in piezometric head can be expected in such a basin.

Further interpretation of consolidometer testing may provide important geological data on the magnitude and timing of tectonic movement. The method is probably also applicable to forecasting subsidence in oil and gas basins.

Conclusions

All predictions of future changes in global population indicate its rapid increase in the forthcoming decades. Such an increase will unavoidably be associated with growing demand for water, crude oil, gas and other natural resources. Hence, a future acceleration of development of aquifer systems and depletion of oil and gas fields appears to be unavoidable.

Agriculture, one of the main water users in the world, is already well-developed in many humid areas. Its future growth, needed to feed growing global populations, will most likely concentrate in warm, semiarid and arid regions. Experience obtained in California, Arizona, New Mexico, Texas, and elsewhere demonstrates the enormous agricultural potential of semiarid and arid regions. Such regions are particularly attractive to agriculture because of an abundance of sunshine, a long growing season and the absence of damaging precipitation.

Water in such regions may be obtained either locally, from underground sources, or by importation via long conveyance canals. Development of available ground water is, in the short run, less expensive than importation which requires construction of dams, canals, and pumping plants, i.e., large capital investments. Due to regional aridity, the recharge of available local ground water will soon be more than offset by pumping, resulting in an overdraft. Flat terrains, the choice lands for agricultural use, are usually associated with structural basins, which are frequently vulnerable to subsidence. The association of irrigation canals and subsidence is, therefore, not an accident. Proper understanding of origin and extent of subsidence is, therefore, mandatory for correct data-gathering, planning, and design of new, and rehabilitation of old, engineering projects. Genetic classification allows relatively easy "fingerprinting" of subsidence, while the concept of an endogenic framework of exogenic subsidence seems to allow relatively inexpensive detection of aquifer systems susceptible and nonsusceptible to subsidence. This concept may also be applied to predictions of possible amounts of future subsidence. The same approach will also probably be applicable to oil and gas fields.

Geologic interpretation of consolidometer testing may also be helpful in understanding the past tectonic history of an area. For example, such an evaluation allowed tracing of a generalized map of Holocene uplift in the west-central portion of the San Joaquin Valley (Prokopovich, 1983). Knowledge of the existence of such neotectonic movement is important for proper survey of land subsidence.

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POTENTIAL FOR SUBSIDENCE FISSURING IN THE PHOENIX ARIZONA USA AREA

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Abstract

Earth fissures associated with as much as 2 m of subsidence caused by water level declines of more than 100 m have the potential for damage to urban infrastructure in the Phoenix metropolitan area. The long, narrow eroded cracks open in alluvium by differential subsidence localized by buried bedrock hills and scarps, the edge of a stable ("hinge-line") or advancing subsidence area, sedimentary facies changes, manmade changes in vertical loading, and groundwater recharge mounds.

In Paradise Valley, gravity and subsurface data indicated a fissure in Phoenix occurred over a buried hill. Nearby similar features and a buried fault may localize fissuring in the basin, including parts of Scottsdale. Fissuring near Mesa has been associated with two shallow bedrock areas, and a similar nearby area may soon also undergo fissuring. No fissures have yet to occur in Deer Valley, however two large buried bedrock ridges may influence fissure formation as subsidence continues.

Introduction

Earth fissures, long narrow eroded tension cracks, have been an increasing problem in south-central Arizona basins over the last several decades as a result of land subsidence caused by a tremendous overdraft of groundwater supplies. Until recently, the fissure-subsidence problem was largely confined to agricultural areas where pumping was most concentrated, but now the problem is also becoming a major concern in rapidly growing urban areas.

Some of the most important parts of the municipal infrastructure are the most vulnerable to damage. The operation of canal, storm drainage, and sewer systems dependent on gravity flow may be impeded because of altered gradients; wells can be damaged by casing protrusion or collapse; and buildings, roads, bridges, and utility lines may be directly damaged by fissuring.

Description and Origin of Fissures

Fissures occur in alluvial basins of southern Arizona where groundwater levels have declined from 45 m to more than 90 m accompanied by generally more than 0.3 m of subsidence. The openings are typically several hundred meters long, 1 m or more wide, and 1 to 20 m deep. Little or no vertical offset is observed; however, there are some notable exceptions. A fissure in the Picacho basin, located about 100 km SE of Phoenix, measures 15.8 km in length where as much as 3.8 m of land subsidence has occurred (Laney, Raymond, and Winikka, 1978).

The fissures begin below the surface as small tensional cracks, later eroded to the surface by heavy rains or application of irrigation water. Runoff enlarges the fissure in length and frequently opens other cracks subparallel to the initial one.

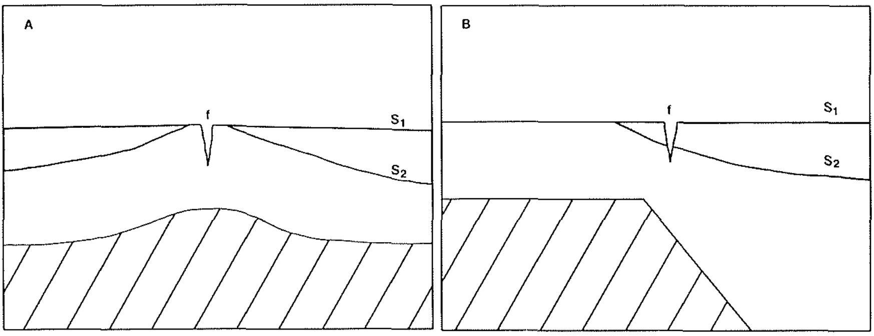
Ground fissures may originate from a variety of processes including: 1) tectonic movements, 2) desiccation of expansive clay soils, 3) hydrocompaction (caused by the artificial wetting of low density sediments or artificial fill), 4) horizontal seepage stress related to groundwater flow toward pumped areas, and 5) localized differential subsidence. The majority of fissures in Arizona basins are caused by the last process (differential subsidence) which is the basis for fissure prediction as outlined in this report. The other processes are of either little importance or are a shallow soil phenomena caused by application of surface water rather than groundwater withdrawal (expansive soil and hydrocompaction). Horizontal seepage stress may in some cases act together with differential subsidence to cause fissures.

In central Arizona basins, subsidence generally begins after water levels decline more than 30 m (Holzer, 1981). With continued subsidence, fissures form at points of maximum horizontal tensile stress near maximum convex-upward curvature of the subsidence profile. The temporal and spatial distribution of fissures is determined by the historical development of water resources, hydrogeologic characteristics, and subsidence history of a particular basin. The term "bedrock" as defined in this study is defined as consolidated rock that is essentially incompressible under the magnitude of effective stress changes resulting from water level decline. Thus, bedrock includes all crystalline rock, indurated conglomerate, and evaporites (anhydrite, salt) that are common in basins of south-central Arizona.

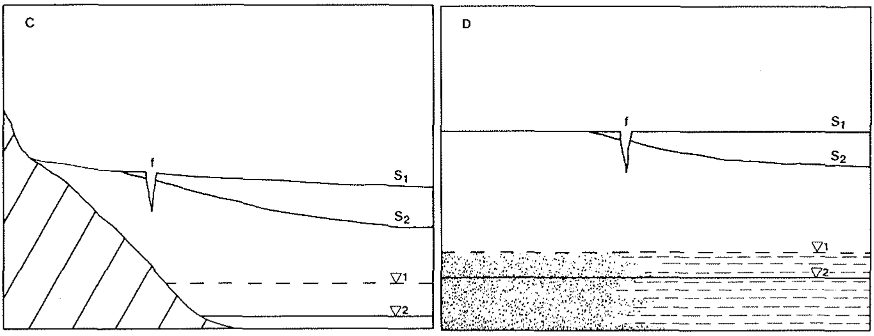
Differential subsidence may be localized by several different geological environments: 1) buried bedrock hills, 2) buried bedrock scarps, 3) the "hinge" or "zero-line" of subsidence, 4) sedimentary facies changes, 5) the edge of an advancing subsidence front, 6) manmade changes in vertical loading, and 7) near recharge mounds. Geophysical and land survey data indicate most fissures are associated with the first three environments, particularly with bedrock hills (Anderson, 1973; Jennings, 1977; Jachens and Holzer, 1979; Larson, 1982a) (Fig. 1). The other environments are of lesser importance, but may explain the occurrence of transient fissures or fissures at the center of a basin.

Subsidence varies as a function of the thickness of alluvium; therefore differential compaction over buried hills (Fig. 1A) can produce a convex-upward subsidence profile with fissuring either directly over the buried ridge crest or offset some distance away from the crest. Buried bedrock scarps (Fig. 1B) of tectonic or erosional origin can also cause a rapid change in the amount of compaction and subsidence within a short distance. Such a subsurface feature may cause vertical offsets on fissures, as apparently has occurred along the Picacho fissure previously mentioned.

Another common environment of fissuring is near the "hinge-line" of subsidence (Fig. 1C) (Larson, 1982a). The location of the hinge-line is delineated by the area where the original depth to groundwater (prior to pumping or application of irrigation water) approximates the depth to bedrock. Little or no compaction and hence no land subsidence should occur where depth to bedrock is less than the original depth to groundwater. At greater depth there is an increasing amount of subsidence. Fissures can form along or parallel to the hinge-line in the subsiding area.



1A. Buried bedrock hill 1B. Buried bedrock scarp

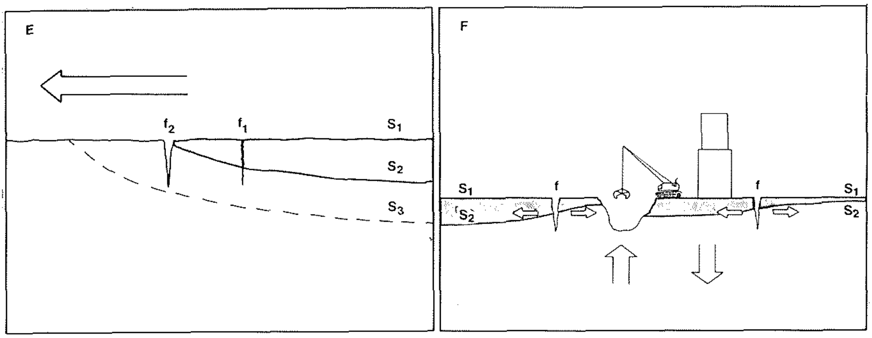


1C. Hinge-line 1D. Sedimentary facies changes

FIG. 1A-D Geologic Environments of Fissuring.

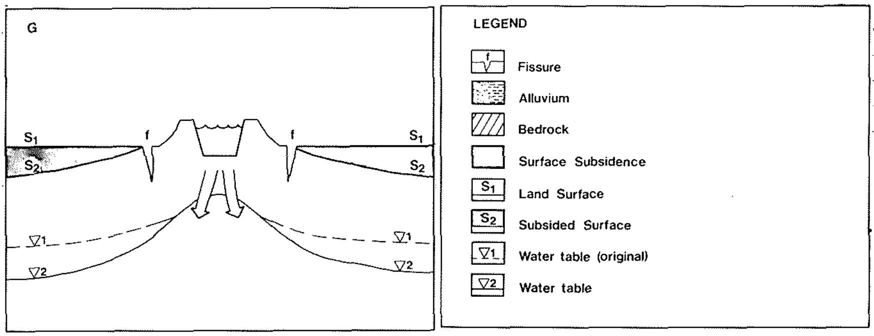
Finer-grained sediments such as silt and clay tend to compact more than coarser-grained sands and gravel; thus differential subsidence and fissuring may be localized by abrupt facies changes in basins (Fig. 1D). The growth of a subsidence "bowl" or area may be rapid enough to cause fissuring along the edge of the subsidence front similar to ground failures which occur over mined areas, particularly for coal (Larry Powell, U.S. Bureau of Mines, personal communication) (Fig. 1E). These fissures are transient, and may close or heal because of the short length of time such areas undergo tensile strain.

Fissures may result from artificial changes in vertical loading or recharge mounds in subsiding areas (Figs. 1F and 1G). Because compaction of sediments from declining water levels is caused by increases in effective stress [difference between total stress (exerted downward) and



1E. Edge of advancing subsidence front

1F. Manmade changes in vertical loading



1G. Groundwater recharge mound

FIG. 1E-G Geologic Environments of Fissuring.

hydraulic stress (exerted upward)] of the deposits, cultural activities that affect either total or hydraulic stress (in addition to groundwater removal) may be important in influencing fissure location. For example, construction activity or fluctuating levels of impounded water behind dams could change total stress, therefore also affecting effective stress and creating loci of differential subsidence.

Potential Fissure Areas in the Phoenix Metropolitan Complex

Earth fissures have been noted in northeast Phoenix, Queen Creek, Luke Air Force Base, and east Mesa-Apache Junction areas where continued fissuring is likely (Fig. 2). In addition, there are at least five other specific areas that can be considered "high risk" for future fissuring where fissures have not occurred (Larson, 1982b). Two of the areas are in Paradise Valley; two in Deer Valley; and one locality is in northeast Mesa.

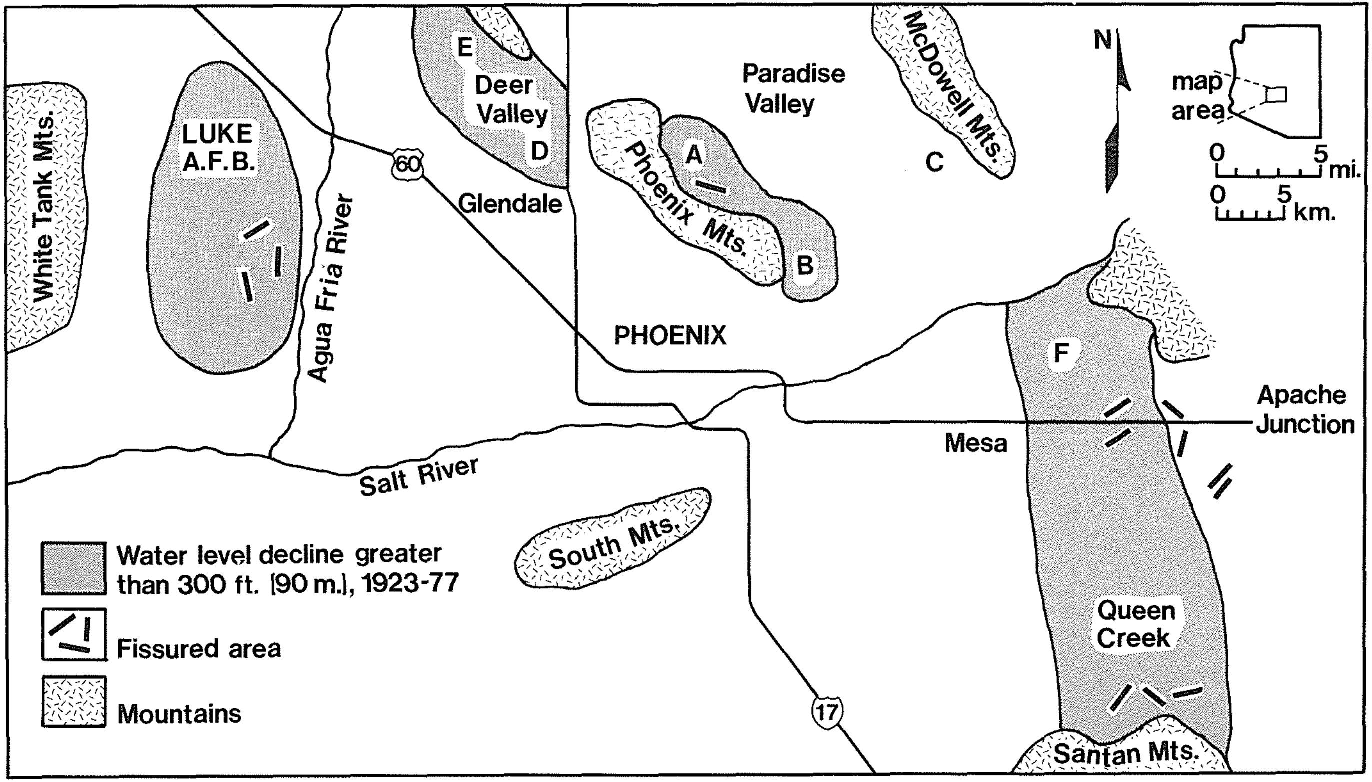


FIG. 2 Present and potential fissure areas in the Phoenix metropolitan complex. (Sources: Laney, Raymond, and Winikka, 1978; Larson, 1982b)

An appropriate procedure for evaluating known or suspected subsidence areas is to conduct a geophysical survey tied to well drilling and other subsurface data to determine buried bedrock configuration (Larson, 1984). The most critical depths for bedrock features are from about 30 to 500 m because the greatest amount of compaction of sediments occurs at this interval as the water level declines. Gravity has been the most effective geophysical method in terms of correlation of anomalies with fissure occurrences, particularly where buried topographic highs exist. Convex-upward gravity anomalies can be generally correlated with a buried convex-upward bedrock surface. At sites where geophysical or other data indicate a high risk of fissuring, land survey measurements to monitor horizontal and vertical ground movements can be used to predict possible fissure locations and subsidence patterns.

In January 1980 the first fissure occurred in the Paradise Valley basin in an area where water levels have declined more than 90 m (A in Fig. 2), and land subsidence of at least 1.05 m has occurred. The 120 m long crack opened in a residential construction site in north Phoenix. Gravity and land survey data indicate the ground failure resulted from differential subsidence over a bedrock hill buried at a depth of approximately 45 m. The hill is part of a series of hills on a pediment that trends NE away from the Phoenix Mountains (Larson, 1982a; Larson and Péwé, 1983).

A large cone of depression of water level decline is directly east of the Phoenix Mountains in Scottsdale that may also be an area of future fissuring (Cordy, Holway, and Péwé, 1980) (B in Fig. 2). Lausten (1973) indicated fissures may form in areas of steep gravity gradients over inferred buried bedrock scarps in Paradise Valley. Land survey data indicated measured subsidence of less than 0.3 m for the area (Arizona Department of Transportation, 1981), however, continued monitoring by the U.S. National Geodetic Survey and the city of Scottsdale will probably reveal even more subsidence has occurred in the area.

Another potential fissure area in Paradise Valley is south of the McDowell Mountains (C in Fig. 2). A prominent convex-upward gravity anomaly in this area of suspected rapid subsidence may represent an area of high potential for fissuring (Larson, 1982b). Because of the proximity of the Central Arizona Project canal (designed to transport water more than 300 km to central Arizona) and rapid urbanization of this area, more detailed investigations should be conducted.

Groundwater level declines greater than 90 m have occurred directly west of the Phoenix Mountains in northwest Phoenix and north Glendale (Deer Valley) north of the Arizona Canal (D in Fig. 2, and Fig. 3) (Larson, 1982b). Fissuring has not yet occurred in Deer Valley. Reconnaissance gravity and well data indicate an area covering more than 60 km^2 where two large NW-SE bedrock ridges are buried at depths less than 300 m that may result in fissuring. Fissuring is also likely south of the Hedgepeth Hills where water levels have dropped over 90 m over a broad area (E in Fig. 2) with as much as 0.14 m of subsidence (Arizona Department of Transportation, 1981).

Since 1963, fissuring has occurred in the east Mesa-Apache Junction areas where water levels have declined more than 120 m accompanied by as much as 2 m of subsidence (Laney, Raymond, and Winikka, 1978; Arizona Department of Transportation, 1981) (E in Fig. 2). Fissures are localized near two gravity highs associated with small isolated bedrock outcrops at Double Knolls and Hawk Rock (Fig. 4). Fissuring has occurred over buried bedrock hills and scarps at both of these locations as determined by

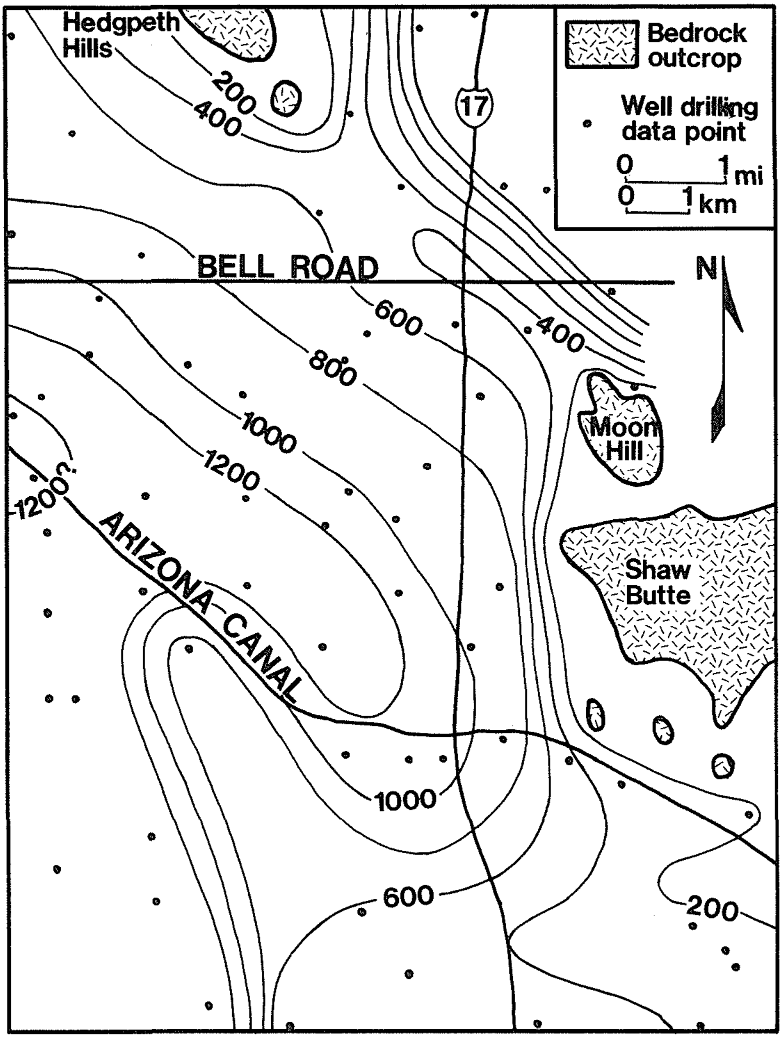


FIG. 3 Depth to bedrock and potential fissure areas in Deer Valley, north Glendale and northwest Phoenix (areas D and E in Fig. 2). Contour interval 60 m (200 ft). (Source: Larson, 1982b)

geophysical and drilling data (Richard Raymond, U.S. Bureau of Reclamation, personal communication). A third gravity high in the same area of water level decline correlates with a mountain buried at a depth of 200 m near Falcon Field in northeast Mesa (Larson, 1982b). As subsidence continues in this area the probability of fissures will greatly be increased.

Conclusions

Officials and the general public should be made aware of the value of geological and geophysical studies with the subsidence-fissure problem.

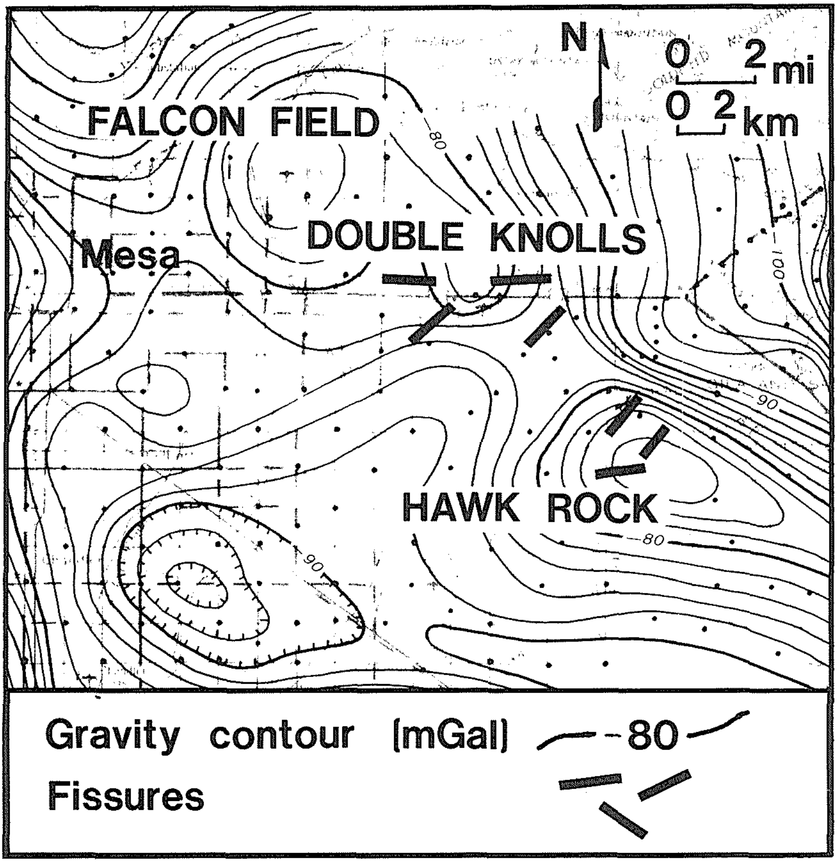


FIG. 4 Bouguer gravity, fissures, and associated bedrock outcrops in east Mesa basin. Gravity contour interval 2 milligals.
(Source: Peterson, 1968)

The objectives of these investigations to assess and alleviate problems should always be within the context of a much broader problem--the problem of conserving a limited water supply in an arid environment.

Acknowledgments

Information provided by the staffs of the U.S. Geological Survey, U.S. Bureau of Reclamation, Arizona Department of Water Resources, Arizona Department of Transportation, City of Phoenix Water Production Office, and the City of Scottsdale Engineering Department was most helpful. Appreciation is due to Allen Moody, Department of Geology, Fort Hays State University for his drafting work and thoughtful review of this paper, and Lynn Vogler, also of Fort Hays for insights on subsidence problems. Gene Rohr and Bryce Bickford, Hays Daily News provided technical and duplicating assistance. Gratitude is also extended to Jeanie Michaelis and Mrs. Harold Chambers of Hays for typing of preliminary and final copies of this paper.

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WAYS TO PREVENT THE EFFECTS OF LAND SUBSIDENCE ON COASTAL CITIES: THE VENICE CASE

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Abstract

Man-induced and natural land subsidence along with the eustatic raising of sea level are the main causes of the increase of "hight water" that has recently been observed in Venice.

There are many ways to defend land from flooding such as: i) general up heaval of area; ii) intercepting tidal flow to control water level in the area to be protected; iii) protecting some particular zones of this area by raising the edges of each part involved. Each method requires a different study, cost ad time for its effectuation and at the same time offers different benefits.

In order to defend the Venice lagoon from the recurring "hight water", the Italian Public Works Department has recently approved the solution of building movable sluice gates at the three lagoon mouths. It will be possible in this way to cut off tides only when the "high water" overcomes a pre-fixed value. At the same time it can permit the free exchange of water between lagoon and sea and allows normal port activities as during normal tides.

This solution would require additional interventions to defend those areas of the city with altimetrical levels lower than the one choisen as the "safety hight water mark". These areas are subjected to the effect of those floodings so called "mean high-high water". One must notice that as a conseguence of land sinking now the city begins to be flooded at a water level of 0,80 m above m.s.l.

To solve this problem the autors propose a solution that consists in grouping some of the islands, which make-up the city of Venice, and intercepting the canals, that separate these islands, with movable sluice gates. In this way the canal network making-up each group unit of island can be completely separated from the lagoon water keeping the water under control within the mentioned network.

Moreover this solution gives the possiblility to protect the city without additional work on the sewage sistem.

The suggested intervention, which includes the raising of edges and waterproofing of walls, offers the adwantage of a reduced impact on urban life either during the actual construction or when in function.

At present this plan considers the division of the city into 18 island groups and an experiment on one of these groups considering the sensibility of the subsoil structure to seepage and related problems, i.e. water pollution.

General

Man-induced and natural land subsidence along with the eustatic raising of sea level are the main reasons for the increase of the "high water" that has recently been observed in Venice.

There are many ways to defend land from flooding such as: a) general up- heaval of the area; b) intercepting of the tidal flow to control water level in the area to be protected; c) protection of same particular zones of this area by raising the edges of each part involved. Each method requires a different study, cost and time for its appliance and at the same time offers different benefits.

The choice of the most suitable method depends on many matters, keeping in consideration each different aspect of the actual problem such as geological, morphological, ecological, economical, financial and sociological reasons to which the best solution is linked.

The first method "up-heaval of the areas involved" can be obtained in two different ways: either by loading with brand new material the area to be up-heaved, or by injecting suitable mixtures in order to let the ground level raise up to the required heigth.

The first solution can unfortunately only be applied to restricted areas, due to the intensive and important building of the venetian territory; the second solution, although very good promising for the chances given at present, requires, before being applied to the weak venetian surface, further deep studies and experiments.

The other methods, already widely applied in other areas for different reasons (Netherlands, Great Britain, etc.), are the ones which met more approvals.

Problem of Venice

In fact, in order to defend the venetian lagoon from the frequent floods, the "Public Works Board" has recently approved the solution of building moving sluice gates at the three lagoon mouths.

It will be possible in this way to cut off tides only when the "High water" overcomes a prefixed heigth. At the same time it can permit the free axchange of water between lagoon and sea and allows normal harbor activities just as during normal tides. This solution would require additional interventions to defend those areas of the city with lower altimetrical levels than the one chosen as the "safety hight water mark".

These areas are subject to the effect of those floodings so called "mean high-high water". One must notice that as a consequence od land sinking, the city begins now to be flooded at a water level of 0.80 metres.

These minor floods are the ones included between 0.8 and 1.1 metres, this latter chosen for the closing of the harbor exits according to the Study Plan of the Public Works Board. They are floods much more frequent than the exceptional ones and therefore not to be neglected.

The necessity of these interventions, also underlined in the approval vote of the Council of the Public Works Board for the study Plan, springs out from the intention of reducing the closings of the harbor exits, to be done only in the most dangerous floods which are less frequent with acceptable consequences on the exchange of water between sea and lagoon and on its pollution, as well as on the efficiency of the harbor activity. This aspect of the problem, which foresees the up-heaval of the outlines of the city up to the level of 1.1 metres indicated as the one chosen for the closing of the harbor-exits, in the reason for the choice made by this group about the defense level of 1.2 metres plus other 20 centimetres of safety, keeping in consideration the future sinking of the town and the water-raising due to the wind.

This group has examined the possibilities of intervention "island after island" which include, as already mentioned, the up-heaval of the outlines of each isle of which the city itself in composed of, as well as the restoration and waterproofing of the city sewer system in order to prevent the flood from coming up through them, and also the check of the outline walls so that the infiltrations can be stopped; moreover the collecting and the unloading of the rains.

The examination of this solution has demonstrated its possibility to be applied even if bound to many difficulties of technical and economical kind and above all to the respect for the venetian architecture. The uprising of the pavements on the banks cannot always be applied as it could cause some sensible modifications to some local characters.

The intervention on the outline walls is very expensive considering their length of about 120 kilometres. The intervention on the sewer system could mean the destroy and the re-building of the pavements of many areas and house ground-floors.

This group has therefore studied a different solution consisting of the interception of the canals in the area to be defended with moving sluice gates, so that the water may not overcome the flooding level of the city. The area to be defend can therefore not only be one island but a group of isles whose outline must be chosen keeping in consideration two main results: obtaining a restricted extension of the openings to be protected, in direction of the outer canals and a small amount of canals to be intercepted.

In this case the intervention should include: some pumping stations in the inner canals in order to draw the sewage and rain waters into the outer canals; all this in addition to all the other interventions of the "island after island solution". Of course this would mean a shorter outline length to work on than the one of each single isle. This kind of solution has many different advantages; the reduction to a few cases of defense from flooding of long parts of banks and consequent better and longer respect for the venetian characters: a shorter outline development on which the intervention should take place (around 35 km) with re-building and waterproofing of the supporting walls and the restriction to the only existing cases of problems due to the sewage exausts around the outline of each group of islands; therefore less waterproofings, less watertight gates and consequently a safer and more economical system.

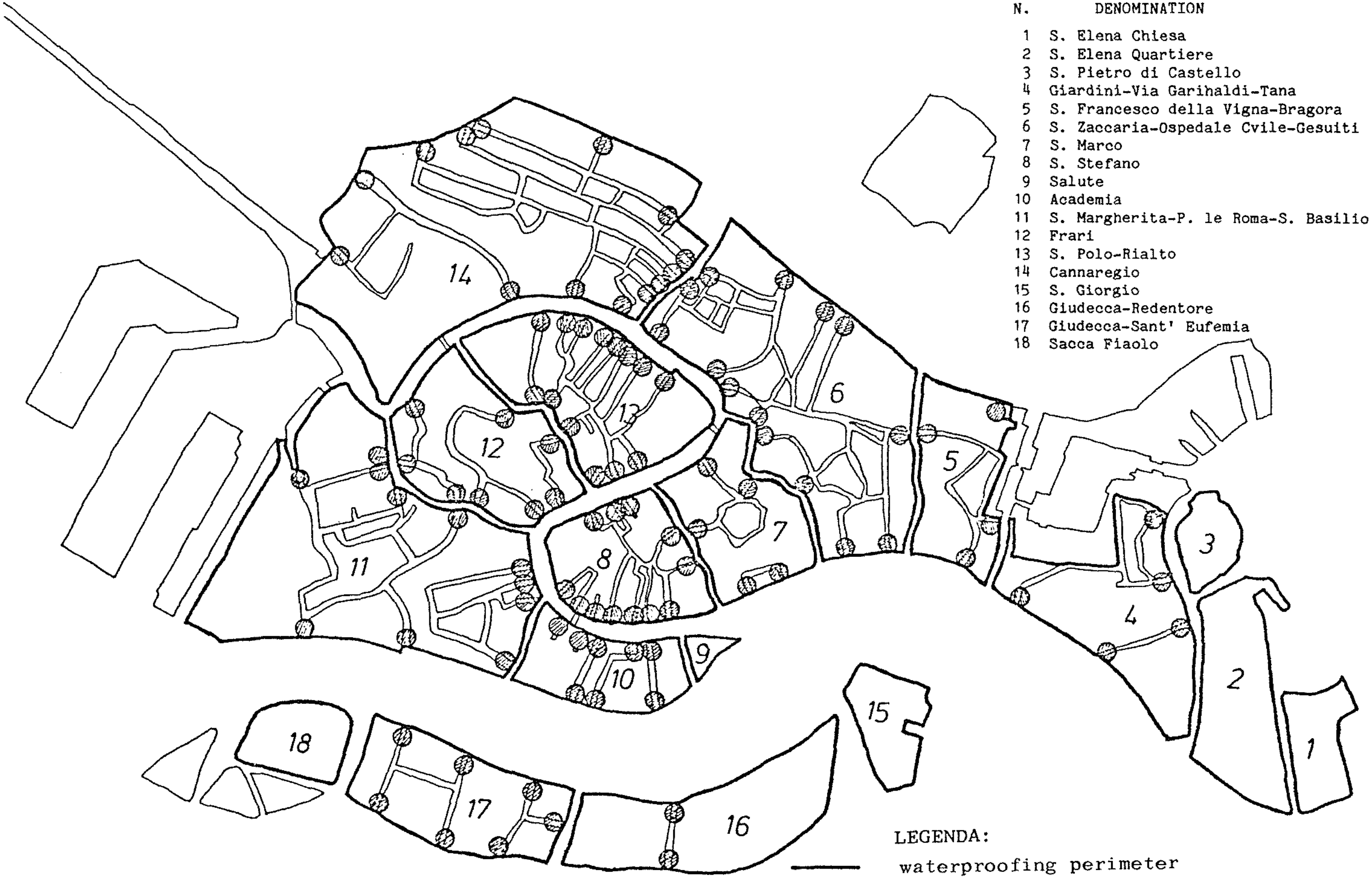
The supposition of dividing the city into 18 groups of islands (see fig. 1) could eventually be revised after some studies and experiments on one of then. For this purpose the island (see fig. 2) has been chosen because of the different characters, of the high density of population and of the damaged buildings. After considering the different proposals and the different expenses and benefits for the final appliance of the best solution for the whole city, the following measures have been chosen: installing sluice gates on the exits of the inner canals (see fig. 3); elevation of the bank-pavements where permitted by the local characters or otherwise the installation of moving sluice gates in the places of possible flooding (see fig. 4) in order to be able to temporarily elevate the flooding level as shown in fig. 4; interception and restoration of the city sewerage; restoration and waterproofing of the outer foundation walls.

The description of all measures is published with detail in the project of May 1982. We wish now to point out that all gates remain completely hidden when not operating so that the venetian characters are not spoiled. The carrying out of the whole project to all the city requires a modest expense, around 135 billions it. Lire at 1982 rates, that is to say 15% of the global intervention for the regulation of the harbor exits of which this plan is a complementary part.

Conclusion

On the other hand, we have the following advantages:

- low penalties to the harbor activity and economy due to the shutting of the exits, consequent to the chosen flooding-defense-level which gives sufficient warranties for about 50 years;
- better exchanges of waters between sea and lagoon with positive effects on the lagoon pollution;
- definite improvement of the buildings conditions, as the flooding



N.	DENOMINATION
1	S. Elena Chiesa
2	S. Elena Quartiere
3	S. Pietro di Castello
4	Giardini-Via Garibaldi-Tana
5	S. Francesco della Vigna-Bragora
6	S. Zaccaria-Ospedale Cvile-Gesuiti
7	S. Marco
8	S. Stefano
9	Salute
10	Academia
11	S. Margherita-P. le Roma-S. Basilio
12	Frari
13	S. Polo-Rialto
14	Cannaregio
15	S. Giorgio
16	Giudecca-Redentore
17	Giudecca-Sant' Eufemia
18	Sacca Fiaolo

LEGENDA:

—	waterproofing perimeter
▨	sluice gates

FIG. 1 City of Venice - Delimitation of isle groups.

EXPERIMENTAL AREA

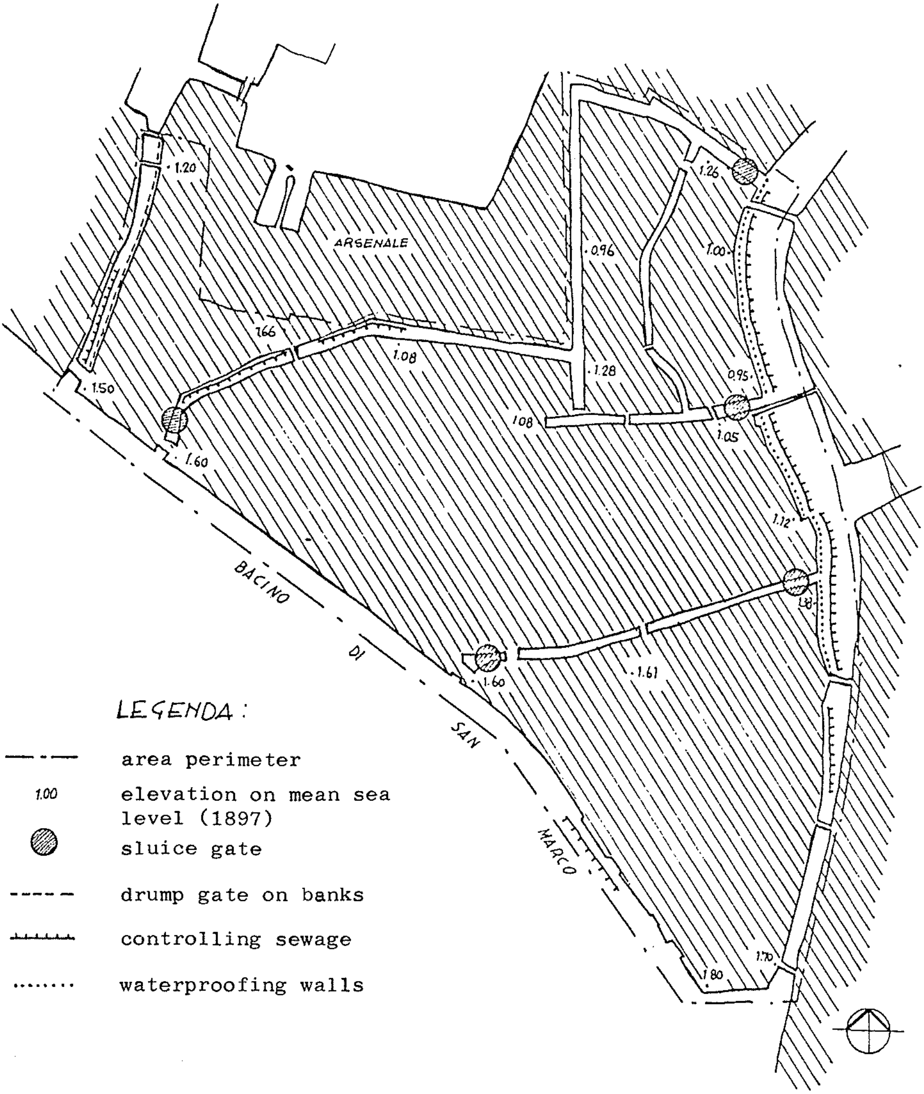
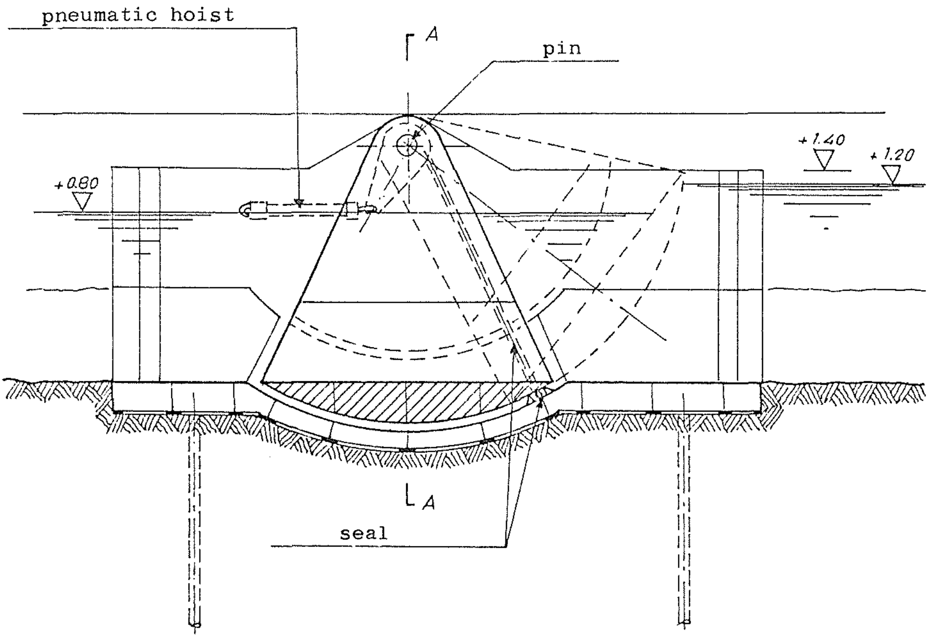


FIG. 2 Experimental area

SBARRAMENTO CON PARATOIA A SETTORE A SCOMPARSA



SEZIONE A - A

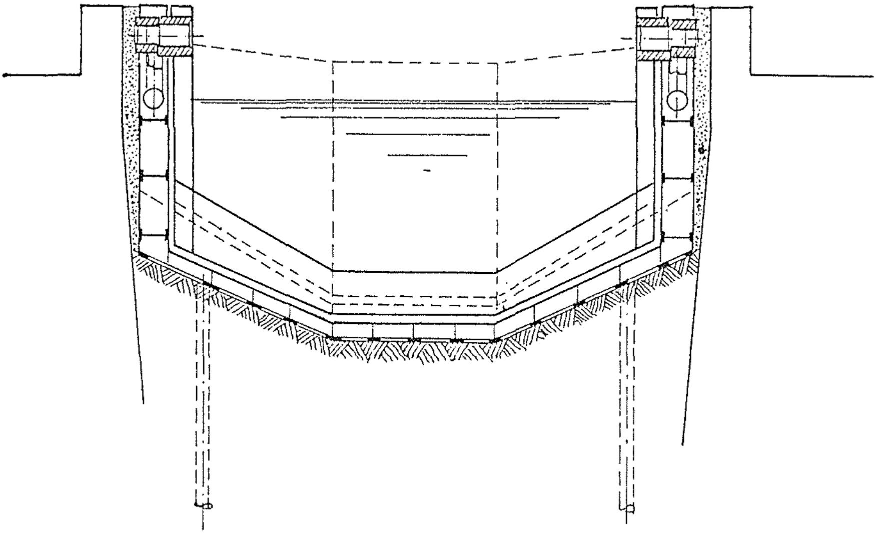


FIG. 3 Submerged tainter gate

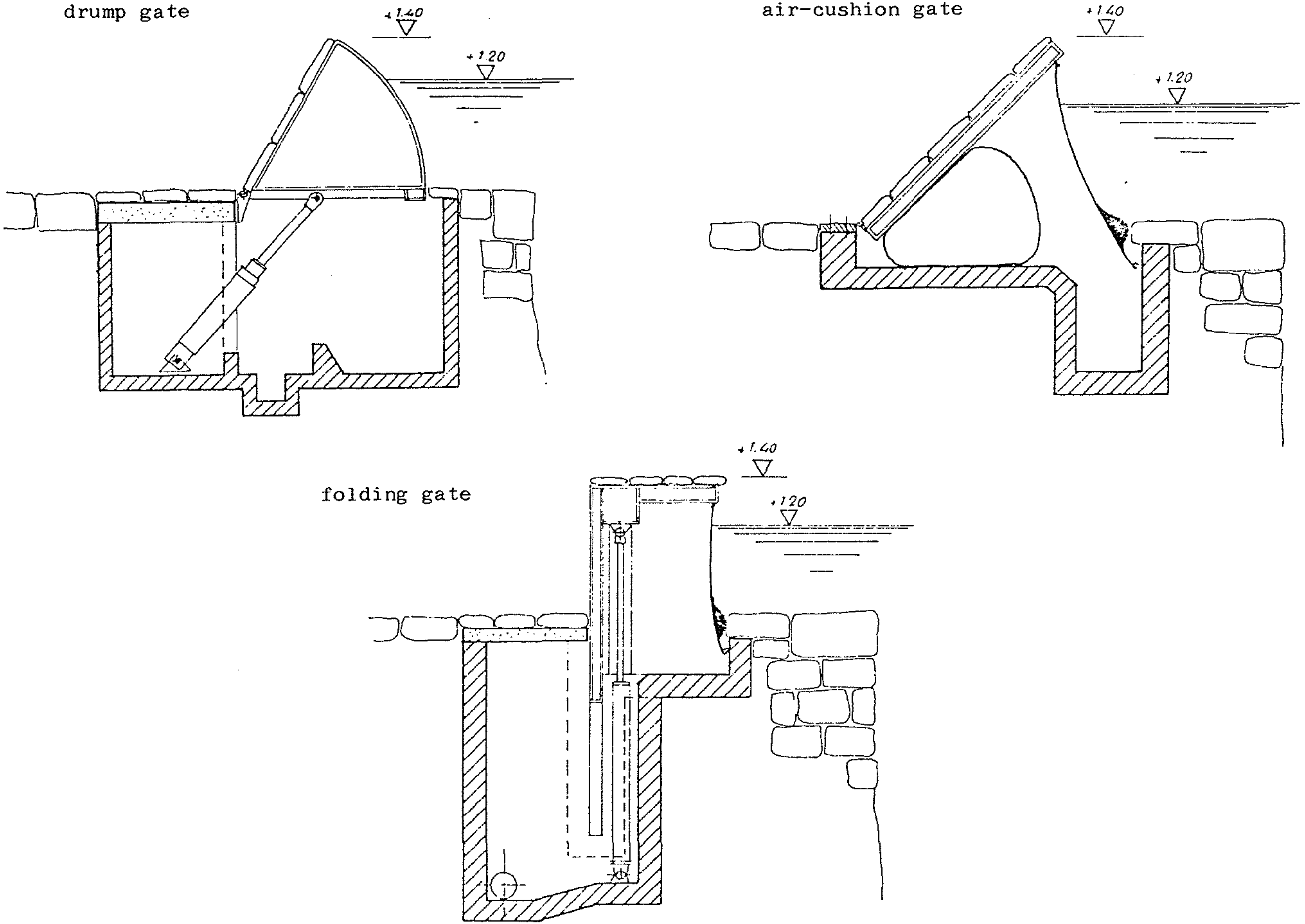


FIG. 4 Some solution to bank gates

level of the city would constantly be fixed at 0.8 metres with consequent less wetness of the walls;

- the foreseen interventions, including the restoration of the foundation walls, would also be the beginning of the general restoration of the whole city;
- easy intervention and maintenance of the inner canals;
- this plan allows the systematical sperimentation and data-collection during its building-up and use, which is, of course, very useful for the study of the problems connected with sewerage and walls.

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LAND SUBSIDENCE IN THE DELTA AREA RIVER PO: DAMAGES AND REPAIRING WORKS

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SUMMARY

The entire Polesana area between the Adige, the Po and the Adriatic Sea has been affected by a noticeable and abnormal sinking of the soil.

The results have been disastrous especially for the hydraulic system of the land. The land concerned is very young, saved from powerful fluvial waters and protected by hundreds of kilometres of embankments.

To rectify this serious condition, provoked by this subsidence, and also bring back a certain security and efficiency of the reclaimed land, it was necessary to carry out a number of works which as yet have not been finished.

Land subsidence in the Po Delta

The eastern side of the Padana plain, south of the Adige, the entire Po Delta and areas which cover a surface area of about 100,000 acreas between the 1950's and 1960's have all been subjected to considerable land subsidence.

The strongest hypothesis attributed to this phenomenon is the extraction of water and methane gas from the land. At that time, this operation highly interested the Po Delta inhabitants. From 1954 to 1958 about 230 million cubic metres were extracted, followed in 1959 with an extraction of 300 million.

It is not the aim of this paper to deal with the causes of the phenomenon, it is suffice to mention certain significant parametres; specific studies on the causes are presently being carried out elsewhere. Here, it is intended to put into evidence the effects that this phenomenon has had on the delicate hydraulic system of the land.

In numerical terms the sinking has reached average values of 2,00÷2,50 metres. There is not sufficient information on this phenomenon at its earlier stages. Prof. Eng. Mario ROSSETTI, from information based on hydrodynamic altitudes of the Canalbianco and Volta Grimana in 1946, believes the beginning of the sinking goes back to the early of 1949. It seems, though, that the phenomenon had already been noticed during the war years.

The first macroscopic results were revealed in 1953-54, but only from 1957 were accurate levellings recorded (in six-monthly periods) by means of an extensive number of bench-marks distributed throughout the territory. The levellings continued up to 1967 with the same intensity but less frequency than before.

A precise and complete connection with the preceding situation at the beggining of the phenomenon is not easy, perhaps impossible. Many bench-marks quoted at that time no longer exist or have altered considerably. Some, that have been discovered are recorded in the following table.

	1924	1953	1957	1959	1960	1963	1967	1970	1975	1978
Polesella Cs 7/28		7,07						6,25		6,20
Guarda V. Cs 1071			5,03	4,82	4,66	4,34	4,28		4,17	
Villanova M. IGM Cs 34	3,83		3,28							2,70
Papozze IGM Cs 39	3,50									1,75
Mazzorno Sx IGM Cs 55	2,45		1,73						0,86	0,81
Ariano P. IGM Cs 50	3,13	2,53	2,05	1,77				1,21	1,20	1,14
Cavanella Po Cs 1097	1,76		0,98						0,11	0,02

However, the major sinking values were recorded between 1957 and 1962, the year in which the methane-producing pits had to be closed down. The phenomenon began to stop, confirming the validity of the hypothesis. The curves which follow, based on a study by Engineer Lamberto SORTINO give a fairly significant indication on the course of the subsidence between 1958 and 1967.

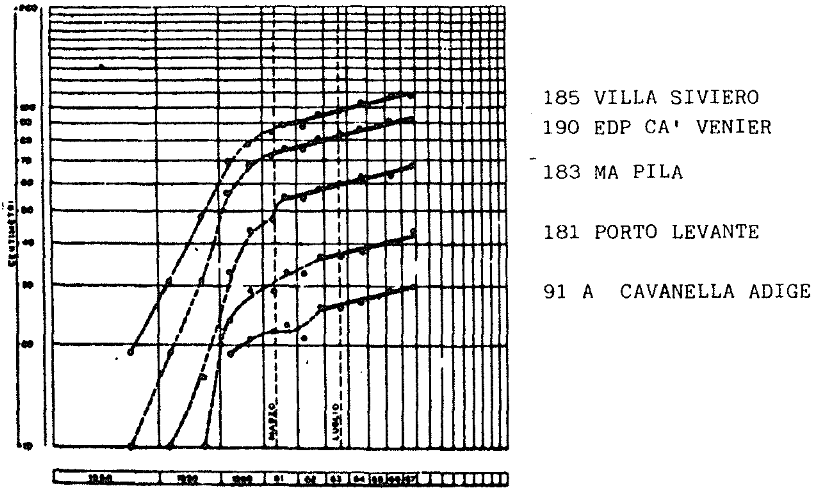

Figure 1

In qualitative terms it is said that the phenomonen has not beenuniform thoroughout the territory but it has assumed the classic form of a basin. The basin which has the highest level of sinking has its axis along the Po of Venice between Contarina and Molo Farsetta in the Porto Viro area; other basins are situated along the delta branches.

On the following map, the areas with the same sinking level are shown.

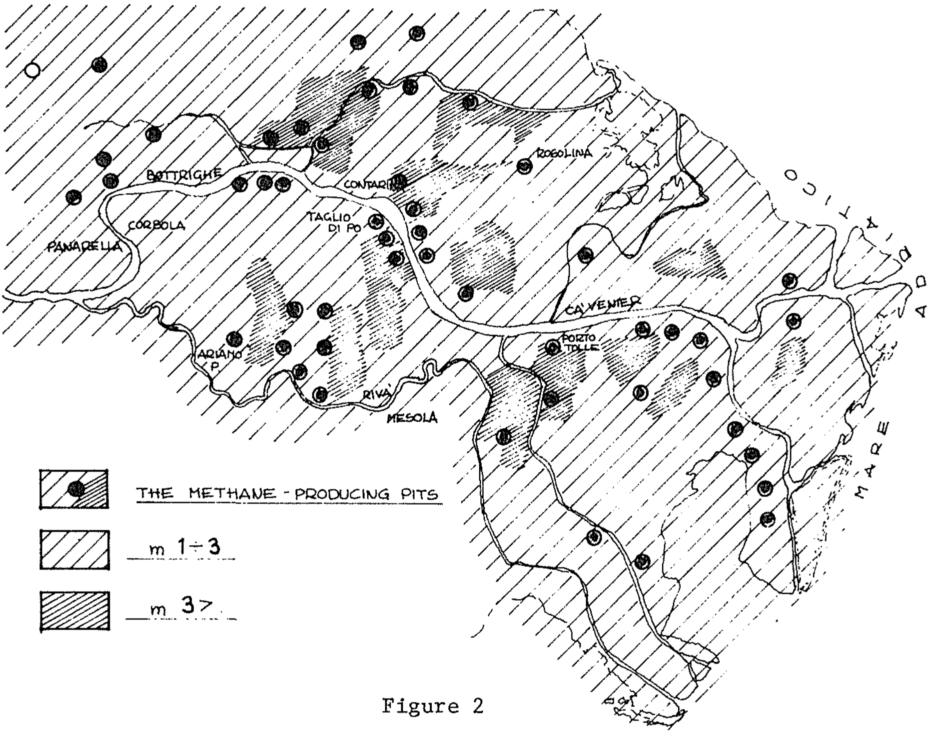

Figure 2

A fairly significant indication of this abnormal land subsidence in the delta area is due also to the examination of the sinkings along the water courses of the area.

As an example, we tive the progress recorded by Prof. Rossetti along the Po of Venice-Tolle-Pila between 1957 and 1962, considering the altitudes noted in 1957 as an initial data.

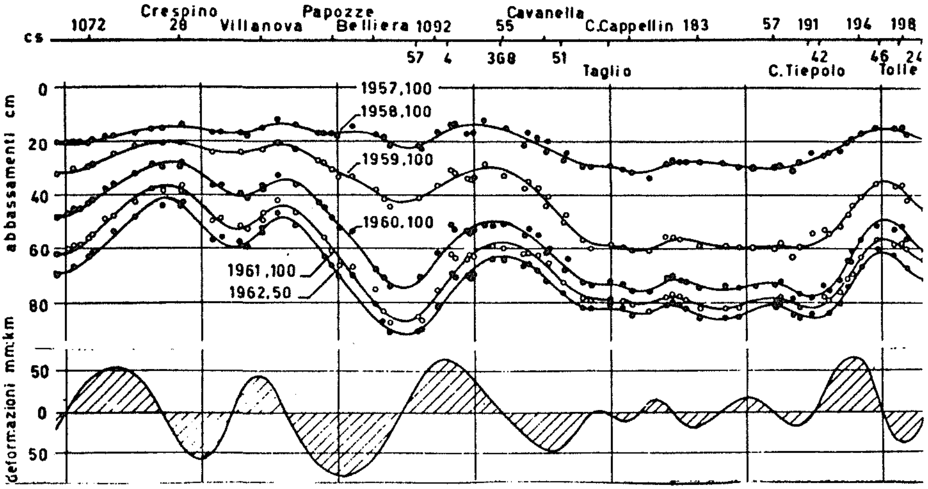

Figure 3

Comments on hydraulic and land reclamation works

The effects of this phenomenon on this territory and especially its hydraulic system have had both progressive and disastrous results. In fact, we deal with young land reclaimed from strong and powerful fluvial waters. The two main Italian rivers the Adige and the Po meet to flow into the sea and the land is crossed by a thick network of land reclaimed canals. In particular, the eastern part, already situated near sea level, with the subsidence came to be completely dominated by rivers and the sea. The territory along the fluvial branches and towards the sea is protected by hundreds of kilometres of embarkments.

The major damages caused by the subsidence are those which have undergone hydraulic and land reclamation works, leading to 3 fundamental factors.

1) An increase of flood lines on country level of all water courses.
2) Coast erosion
3) Alteration of water regimen.

The conseguences of the first factor appeared to be most evident being most striking and macroscopic the area we speak about is hear the sea and slopes slightly. Its water courses which have slow moving currents are regulated by valley and sea conditions. The embankments were not able to retain the floods, due to the highest points becoming lower than the water levels. The hydrostatic charges on the ground level had increased and with them, also the urgency to the embankments.

The underpressured increases in that area tended to be more serious and dangerous: the land has been formed recently, it is not yet consolidated. It is composed mainly of silt, thin sands, and clay with the presence of peaty layers, as shown in the stratographic surveys below .

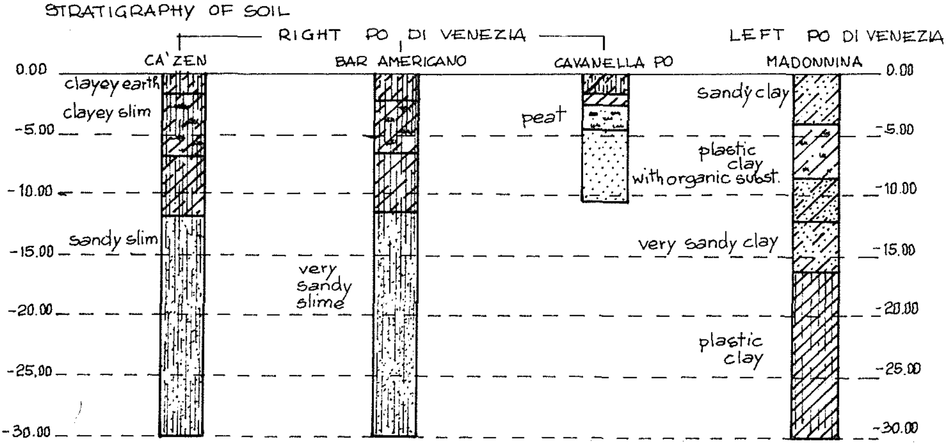

Figure 4

A significant performance of the major dangers on the hydraulic security of the area in the Delta appears evident in the table that follows, drawn out from another study by Prof. Engineer Mario Rossetti 'The breaches of the Po di Goro in June '57 and November '60 and their influence on the moving forces of the Po'. It reports the height of water at its full and the duration in June '57, December '59 and Oct-Nov '60 for certain areas of the Po in Venice.

WATER-GAUGE	June 1957		December 1959		October November 1960	
	H m	Duration %	H m	Duration %	H m	Duration %
PONTELAGOSCURO	3,04	18,8	2,58	28,3	2,83	22,9
CRESPINO	7,30	14,6	6,98	21,4	7,29	15,8
VILLANOVA MARCHESANA	6,78	14,5	6,66	16,3	7,00	10,1
CORBOLA	6,14	15,3	6,30	11,7	6,82	4,5
TAGLIO DI PO	5,02	9,7	5,60	2,2	5,72	1,7

It is also opportune to note that Pontelagoscuro was not affected by the sinking, Crespino and Villanova Marchesana are situated at the western sides of the basin, while Corbola and above all Taglio di Po are situated in the area where the major sinkings have been recorded.

The hydraulic modifications made to terminal stretches of the Po are evident. The altidude of water in the Delta is increasing as the phenomenon advances, with lower depth at Pontelagoscuro, thus increasing the danger of major hydrostatic charges.

In probable terms, it is also noted that the 3 overflows were not ewceptional. In Pontelagoscuro the eventual possibility takes place every 4-5 years, meanwhile, in the Delta the possibility is every 50-60 years.

Much more serious, were certain aspects of the effects had on the subsidence of the marine coast in front of the Delta Padana - a coast composed of a thin beach which has a slight slope.

The solid materials which are carried down to the sea by means of the river, are distributed through the movement of the sea. Thus forming a delta shape according to the direction of winds from the north-east and south-east.

It has been estimated that the rapid sinking between 1949-62 caused the delta coast to lose 500 million cubic metres of solid material, which the river was unable to integrate.

In areas such as the High Adriatic where the tide has theoretical drifts of aroun 1,10 ÷ 1,20 metres, with certain points considerably higher, the action of the sea has no longer had the natural flow of a submerged coast or marine waters. Having destroyed its modest, natural defence system (which before was sufficient to retain rapid sea movement) it flooded valleys and reclaimed land territories.

Thus, the territories of the Delta, already at a low altitude found themselves threatened by the salty sea water: in the cases where they had been flooded, they had to be recovered through the mechanical lifting of all the water and consequently by the 'desalting' of the land.

After stopping the subsidence, the river was able to begin a slow job of coast reconstruction, slowed down by a small amount of solid material coming from the mountains(this concerns other motives which are not applicable to our topic of study).

From the marine shelf opposite the Delta a certain reversibility is noted. Totally irreversible is the effect of subsidence on the internal land the fluvial water regimen, and the system of land reclamation.

In fact, the entire meteoric water system, composed of hundreads of kilometres of canals and more than 40 centres of mechanical lifting, underwent a radical failure due to the inverted inclinations of the canals in certain stretches and for the prevalent increase which the lifting plants had to overcome. We must also add that due to a maior difference between the water levels of rivers and fields and a major volume of filtered water through embankments, there has been an increase in the water capacity to be lifted. In certain basins (not extensive) surrounded by fluvial branches of the delta, the relation ship between volume lifted water and volume of meteoric supply has become 1,2+1,3.

Repairing work

The damaging effects of the subsidence on the hydro-physical system of the Polesine have been pointed out. The disorder, already evident from the disastrous flood of 1951 had also been made worse by the subsidence of the land. In the 1960's the situation seemed so serious that it was considered to be irrecoverable.

The river banks, as already stated were uncapable of holding the waters, due to unsufficient altitude and lack of consistency. The defence system towards the sea was inadequate and could not the energy of the waves - no longer ruined by sandbanks, while the land reclaimed works resulted as being completely devastated. The major preoccupations concerning the water courses of the area, came from the Po Delta and from the belt towards the sea which links the various branches. To give some indication on the works that were necessary in order to repair the serious damages provoked by the subsidence, we refer to those of the Po, and the sea banks border the territory toward the east.

The first works, carried out during the subsidence, had as their aims, an immediate restoration of the bank's system. It aimed to recover the altitude so that it prevented the water overflowing.

It was then necessary to widen the transverse dimensions of the banks to avoid determined siphon from the dangerous underpressures which had been produced. The increased hydrostatic charges (in relation with the already described land traits) were not exceptionally high. There were numerous filter phenomenons, together with an abundance of sandy silt substances.

To obtain such results new flood lines were determined. More accurate controls were also carried out on each single part of the banks by placing a metre mark at the top of the so-called lines. For the transverse dimensions the form of the bank should have concealed the water lines; from the maximum flooding altitude, an inclination of 6 as a base, to one for height would be reached.

All the banks were involved in the above works. Due to financial difficulties the raising and the enlargements were carried out only on certain stretches and consecutive stages on the same stretch of land. It is also necessary to take into account the factor of the banks' barriers being surpassed by the sinking in progress during the years of major subsidence.

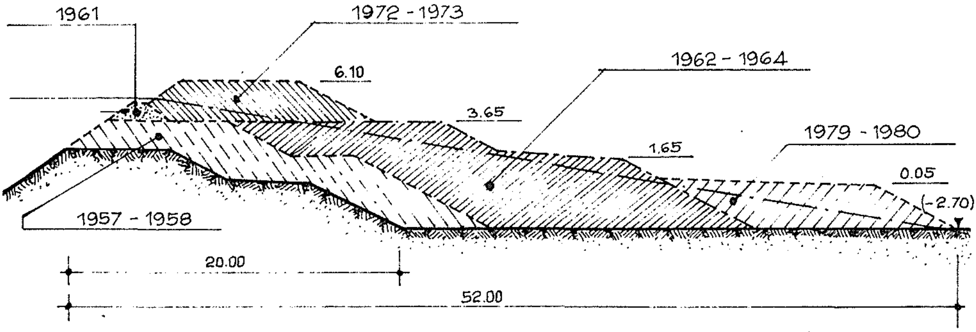

Figure 5

The banks of the River Po and the delta branches in Province of Rovigo extend to over 300 Km: to raise and widen them (a work not yet finished) it was necessary to use 50 million cubic metres of land. To such quantities must be added land needed for territory around Ferrara belonging to the Po Delta. Where underpressures appeared to be more dangerous impermeable diaghrams were carried out. All the road embankments were used for a more efficient service of control and intervention.

To obtain the same level of security along the entire defence system, there are still other works of the same type to be carried out.

Such works though, are passive. An active and substantial recovery of hydraulic security can be achieved through the hydric regimen on the delta stretch of the river.

Certain studies and plans based on a physical model have been carried out. They all tend to seek the best criteria to give the hydrophysic system of the Po Delta a more balanced arrangement and at the same time recuperate what was lost during the subsidence.

Having carried out the most immediate and urgent restoration works, more complete works were begun. They consist of widening particular sections, constructing new banks, the rectification of bends and the elimination of certain parts.

With a more regular layout a better downflow of flood water can be obtained and an effective depression of water lines, thus giving minor hydrostatic charges on the field level.

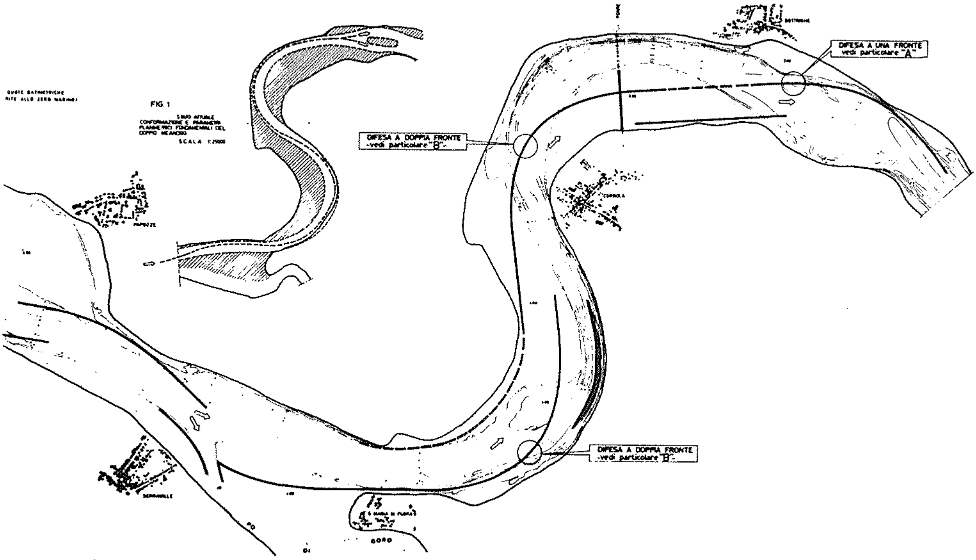

Figure 6

With regards to the sea banks, we must state that before the subsidence, the waves broke onto the front sandbanks: to defend the territory modest embankment were sufficient, which at times simply acted as borders for the fishing valleys.

With the disappearance of the sandy shores it was necessary to provide the closing side of the land towards the sea with robust Shelters. They consist of embankments with altidudes of + 3,50 ÷ 4,00 to m.m. fitted at the front with breakaters; these are especially used on the most beaten stretches.

On many stretches, at a first advanced defence line, another one has been added immediately behind it.

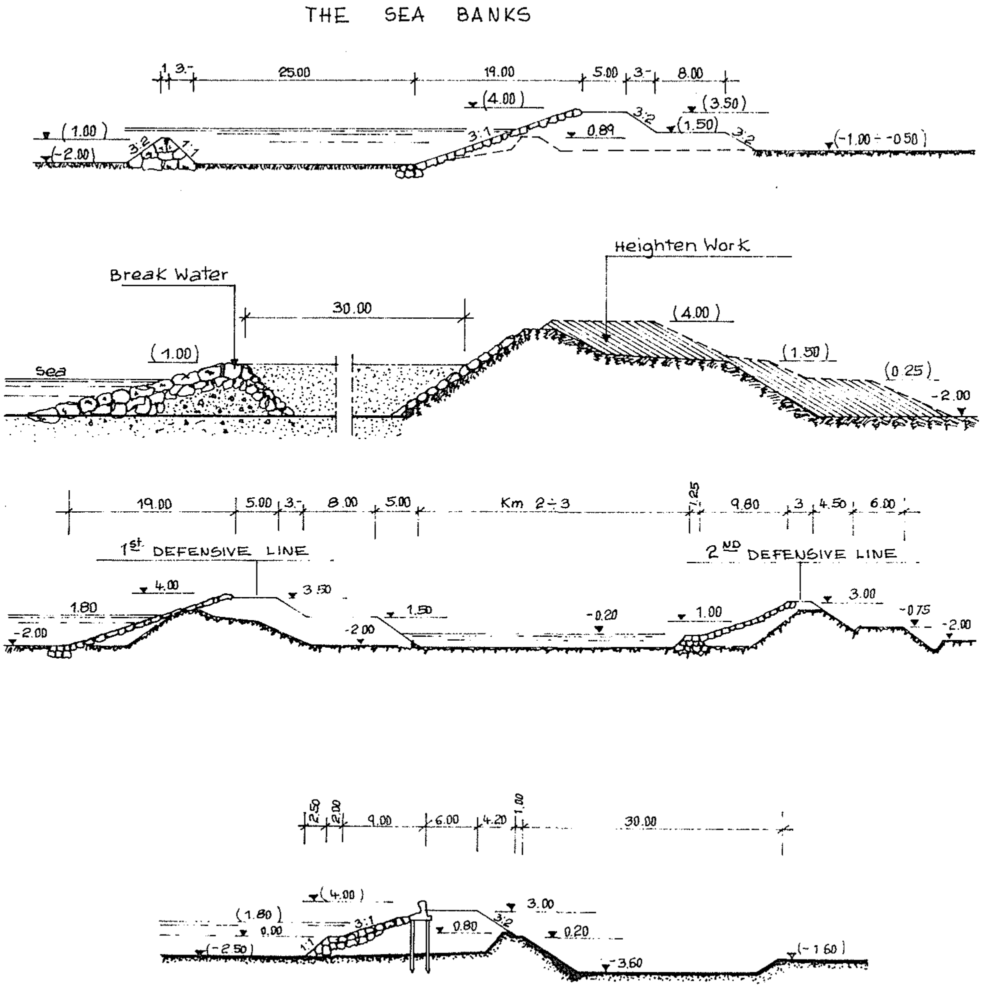

Figure 7

New materials and technologies have recently been adpted to defend the land from the perils of the sea. On coast stretches, where erosion has been most consistent, polythylene tubes filled with sand have been used by means of a netted mesh where moving material (or suspended) can be deposited.

In the majority of land reclamation work cases new planning was used. In fact land reclamation has assumed a new and more serious condition. At present the lifting plants of the Delta area have a lifting capacity of 320 mc/sec for an average of 4 metres. Some have been constructed 'ex noto', others have been maintained and adapted to the situation.

Figure 8

Final Conclusions

The land subsidence of the Po Delta has risked the actual existence of the territory, conquered and consolidated by generations of work with fluvial works and land reclamation works.

But, even after having saved the territory the effects have been such that a modification of the entire layout has been necessary.

It is not out of place to state that in the urgency of intervening in the race against the subsidence of the defence systems, a certain naturalistic parts of the territory were neglected. It is the case that certain damp areas, placed behind the seas defence system have been drained of a more rational defensive organization, this is also the case when modifications on natural surroundings have been felt.

The phenomenon has hit a particular area because of its superficial land and has determined particular situations which will not be repeated, however, the experience will be of future help to similar cases.

One must consider that in treating problems of such vast dimensions, large expenses and much time is needed; each indecision delay and worry has a very high prince.

Therefore, in whatevar way is chosen to save the territory, every other interest must be placed in second place and every reasonable hypothesis must be considered in the shortest time possible.

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AN OVERVIEW OF THE SUBSIDENCE OF VENICE.

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Abstract

A general view of the aspects of the Venice subsidence situation up to the present time, as studied by the CNR in Venice since 1969, is supplied.

A look at the cause and effect relationship is given, along with an illustration of the three factors making-up the land elevation loss during this century (namely, natural subsidence, man-induced subsidence and eustacy).

Finally it is demonstrated that the recent increase in recurrence of flooding is mainly due to the 22 cm of land elevation loss recorded with respect to mean sea level.

The subsidence of Venice is a problem which has come to light in the last decades since the city has been ever more frequently subjected to "acqua alta" (flooding).

Both natural and man-induced responsabilities were hypothesized at the beginning of the '60s for the process of land sinking. However, on the basis of worldwide experiences it appeared certain that the intensive groundwater withdrawals played a fundamental role in the phenomenon (Lofgren, 1969; Poland, 1970).

From 1969 the new "Istituto per lo Studio della Dinamica delle Grandi Masse" (ISDGM), CNR - Venice, included the analysis of subsidence among its research themes, and so a long-term theoretical and experimental, organic an rational research began, specifically oriented to the analysis of this process (Fig. 1). Subsidence induced by rash exploitation of groundwater has been studied in depth, the cause and effect relationship has been quantified and a mathematical model was developed to predict the evolutionary process which was later verified by time (see references and main bibliography) Only a brief review of this occurrence is given here.

The exploited venetian aquifer system included 6 strata from ground level to about 350 meters deep (Fig. 2). The exploitation of underground water performed mainly for industrial use, began around 1930 with modest quantities and increased until it became intensive after the second world war, with the industrial boom. Consequently the hydraulic heads went down with respect to the original condition at the beginning of the century, and the average piezometric decline in the industrial zone was more than 20 meters, 80% of which happened in the critical phase between 1950 and 1970 (Fig. 3). Consequently, the induced land lowering was most intense in this period. In fact, due almost exclusively to natural causes in the first decades of the century, it reached the highest values between 1952 and 1969.

With the 1970 closure of artesian wells and the diversification of water supply in the venetian area, a general improvement occurred quite quickly.

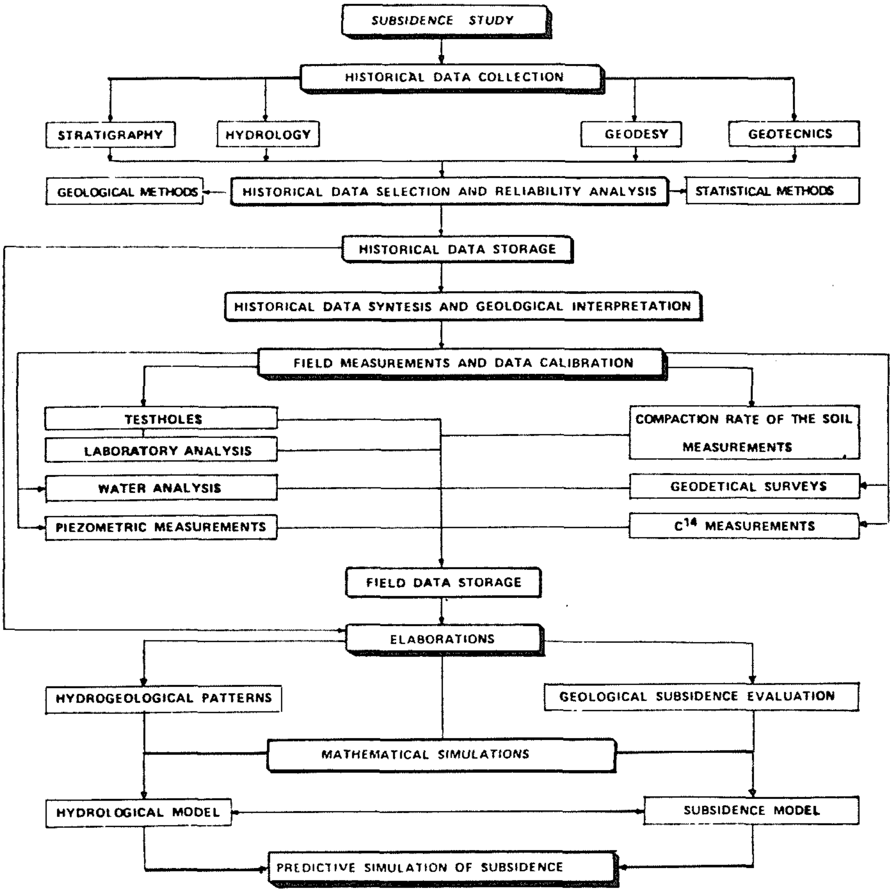

FIG. 1 - Blok diagram of the investigation of subsidence of Venice (after Carbognin and Gatto, 1976).

The hydraulic heads recovered and later rose to ground level, re-establishing the condition which existed before the intensive groundwater pumpage (Fig. 4). At the same time, the ground, which had been in a quiescent period, showed a rebound in 1975, which, in the historic centre, was about 2 cm (Fig. 5); an insignificant figure, but sufficient to show the cessation of induced subsidence.

This positive behaviour of the ground had previously been forseen, under the hypothesis of the closure of the artesian wells,by the mathematical simulation of the subsidence of Venice (Fig. 6); further it was, and is still today, confirmed by the bahaviour of sea level differences (Fig. 7). Comparing the tidal records in Rovigno and Buccari, sites on the Yugoslavian coast which is taken to be stable, in Trieste and those of Venice, until 1969 - that is when subsidence was occurring - the average annual sea level recorded in Venice was apparently increasing with respect to that of the other stations. In recent years this has not occurred any more, and even the rebound appears.

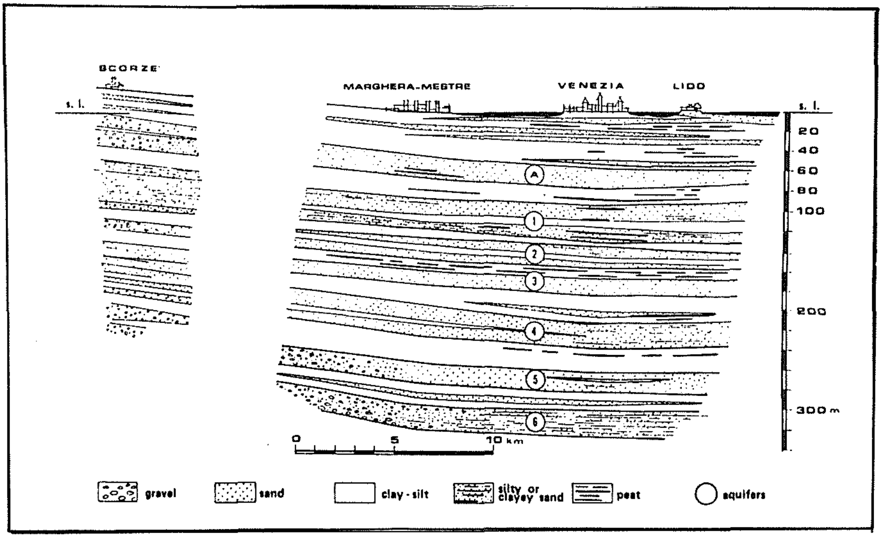

FIG. 2 - Schematic section of the venetian aquifer-aquitard system.The six exploited aquifer are numbered (after Carbognin and Gatto, 1976).

Land subsidence due to groundwater withdrawal is hence no longer a problem today. As long as intensive exploitation will be avoided, the steady present condition will persist.

But a great concern arose after the severe 1976 earthquake, when all northeastern Italy was hit. The geodetic survey carried out immediately after, showed in the focal area notable upheavals which attenuate toward South, in the Venetian Plain (Talamo et al., 1978). Venice itself seemed

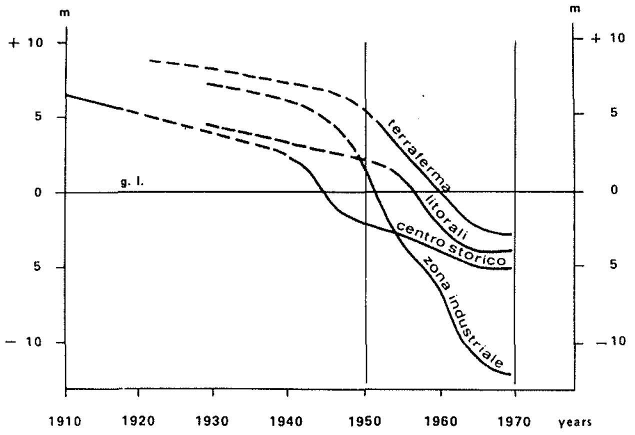

FIG. 3 - Piezometric decline observed from 1910 to 1970 all over the venetian territory (after Carbognin and Gatto, 1976).

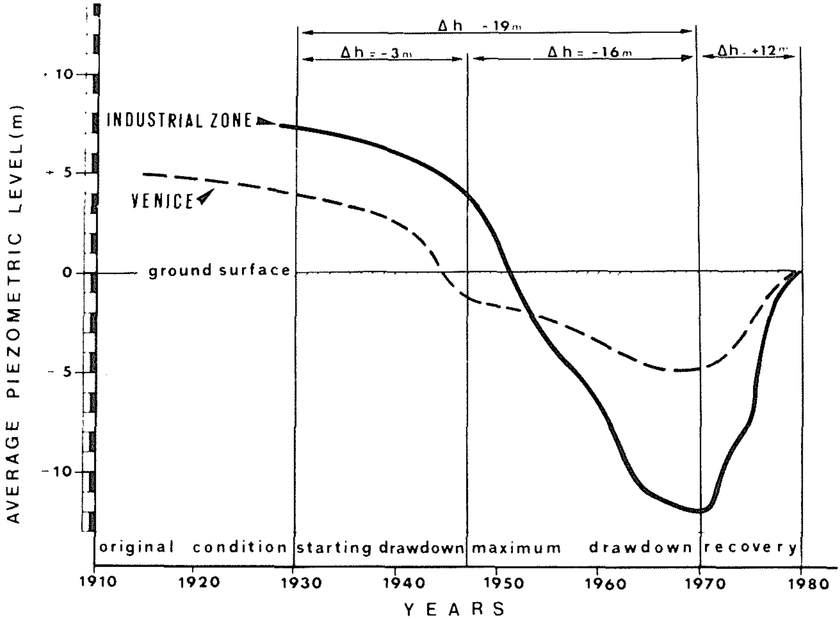

FIG. 4 - *Average piezometric level in the industrial zone* compared with that of Venice. The Δ h values refer to the industrial zone (after Gatto and Carbognin, 1981).

not be involved in the effect: the above mentioned tidal comparison stresses this statement. Next levelling, which is going to be performed in 1984-1985, will further detect possible land elevation desplacements in the venetian mainland.

Notwithstanding the reassuring present situation and the rebound, the 80% of the occurred subsidence is irreversible and its effect still remains.

We must keep in mind, however, that Venice is built in a lagoon environment and it is with respect to water that it must come to terms. For this reason we must remember that natural causes contribute to lowering the height of the city with respect to sea level, even if they are only slight: natural subsidence and eustacy.

Apart from natural subsidence which has been quantified as 0.4 mm/y for the present century, and therefore has a relatively modest incidence on the phenomenon (3 cm from 1908 to today), the process of positive eustacy must not be neglected. The sea level is rising all over the world and also involves the Adriatic and therefore the Venice Lagoon. This process, always assuming a linear trend, has been evaluated as 1.27 mm/y and so it has totalled 9 cm during this century. The combined effect of the three factors (natural subsidence, man-induced subsidence and eustacy) has been quantified as a land elevation loss of about 22 cm from the beginning of the century (Fig. 8). This loss in ground surface elevation, even if only of very modest proportions with respect to subsidence measured elsewhere, both in

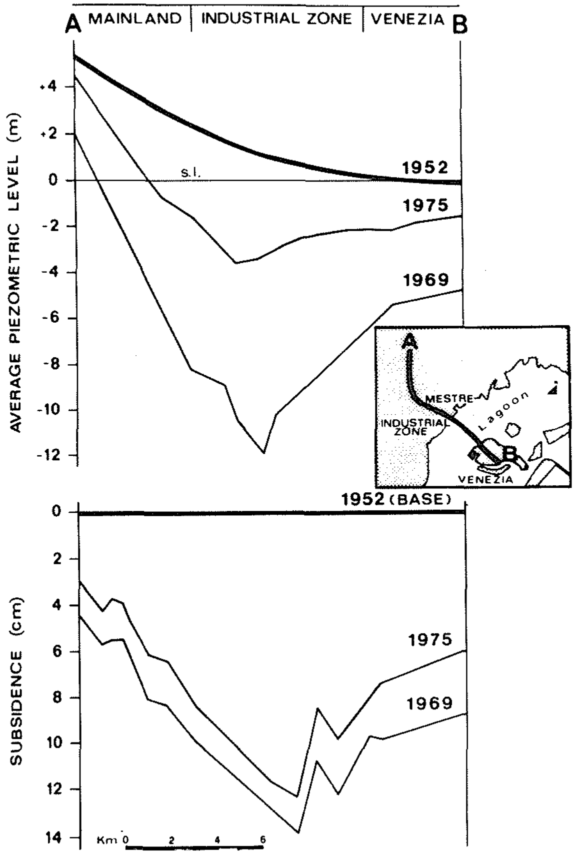

FIG. 5 - Average piezometric level and land subsidence between 1952 and 1975 along a levelling line from the mainland (A) to Venice (B). The close correlation between the two phenomena is evident either in the critical period (1952 -1969) or in the positive stage (1969-1975) (after Carbognin et al., 1976).

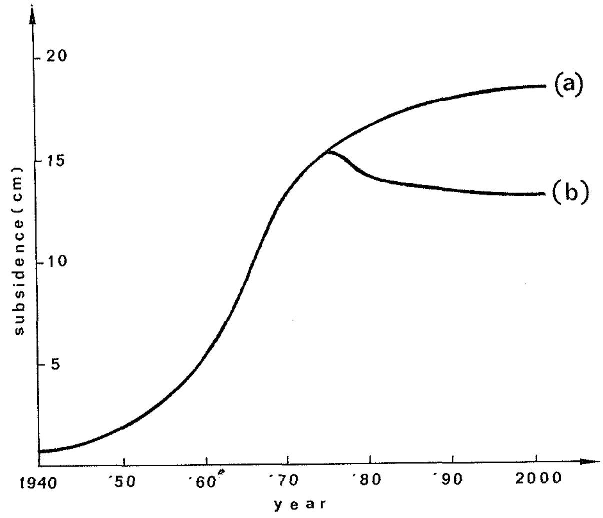

FIG. 6 - Predicted subsidence under the effect of the twe extreme head decline patterns:

a) countinuation of rash exploitation

b) closure of artesian wells

(after Gambolati et al., 1974).

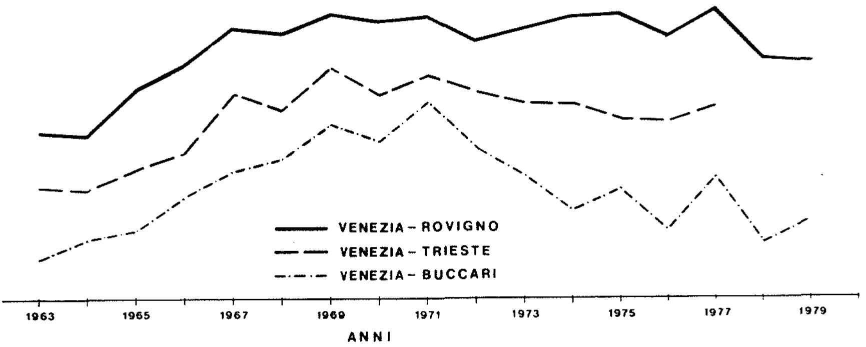

FIG. 7 - Yearly mean sea levels in Venice and Trieste, Rovigno, Buccari: trend of the differences (after Carbognin et al., 1981).

Italy and abroad, has been vital for the Venice's existence, in as much as Venice lives in the water.

In fact, floodings in Venice have undergone a striking increase in frequency, and those tide levels which 80 years ago would not have flooded the city are now included in "acque alte" because of the 22 cm land elevation loss (Fig. 9).

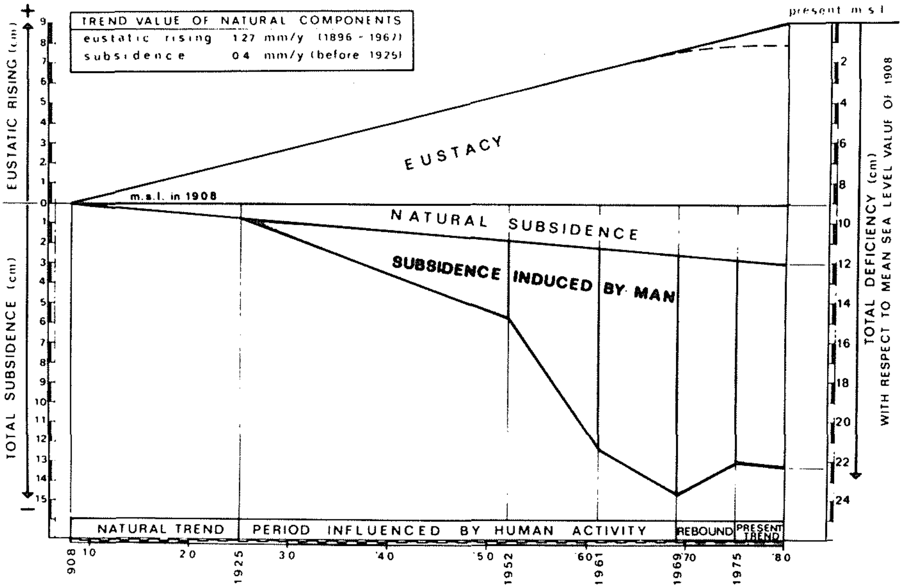

FIG. 8 - The overall land elevation loss of the venetian surface level with respect to mean sea level: the contribution of the three responsible factors - eustacy, natural subsidence, subsidence due to groundwater withdrawal - is indicated from 1908 to 1980 (after Gatto and Carbognin, 1981).

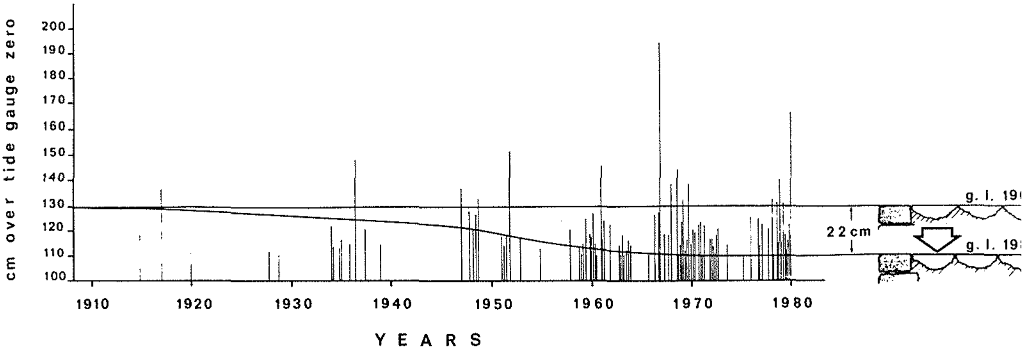

FIG. 9 - The "acqua alta" occurrences from 1908 to 1980. The increase with time is clearly dependent on the loss in ground surface elevation (after Gatto and Carbognin, 1981).

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VENICE TODAY

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Abstract

Based on new hydrogeologic and geodetic measures, the current equilibrium state which the ground attained in the Venice area is briefly illustrated with regards to an existing piezometric improvement and recent seismic movements which affect the region.

Introduction

Flooding, a phenomenon due to meteorological and astronomical factors and consequently connected with the natural laws which govern these episodes, has affected Venice ever since its origin. In fact there are countless documented instances of disastrous floodings which in historical times threatened the very existence of the lagoon city, built on the numerous small, flat, sandy isles.

With the beginning of the fifties, the increased frequency in floodings, involving even vaster city areas, urged a study to understand the phenomenon and look for the causes of this degenerative state.

The mentioned natural factors and the changes in the lagoon dynamics brought about by hydraulic works could not by themselves explain the new phenomenology, which could be reasonably justified only by acknowledging a current and general land sinking. In fact, the 1961 levelings, precisely and accurately carried out, confirm the occurrence of subsidence whose intensity, indeed moderate, was of little damage to areas far from the coast, but fatal for Venice whose altitude is just slightly above sea level.

Studies were immediately begun to identify the causes of this unexpected lowering which was actively occurring in the industrial zone, the historical center and in parts of the Lido littoral, a natural defense for the lagoon against the adversities of the sea. With the participation of all nations, worried about the survival of Venice, unique in the world for its artistic and environmental richness, the subsoil was explored by geophysical and mechanical tests; hydrological and geotechnical characteristics of the soil were identified; and the control network established. Repeated levelings followed the trend of the phenomenon which between 1968 and 1969 reached its critical stage.

When exploitation of artesian wells was identified as the principle cause of subsidence, the use of groundwater was drastically abolished in the worst subsiding areas. In particular, measures were taken to shut down the larger industrial wells, equipped with powerful, submersed pumps, which constituted the main cause of the deep cone of depression that affected Venice and the lagoon islands. As predicted, even with mathematical models, this countermeasure determined very early the natural recharging of the aquifers, and in 1975, the consequent ceasing of man-induced subsidence.

The motive of great concern eliminated, the inconveniences still remain. The phenomenon of flooding is in fact increased by the effects of subsidence which irreversibly lowered the ground level about 10 cm: flood levels are higher, with a greater frequency in floodings of the lower areas and a greater extent in the zones subject to the danger of flooding. Now the physical integrity of Venice is more jeopardized than in the past, while the socio-economic activity of Venice is greatly suffering.

In fact, man-induced subsidence accelerated a natural phenomenon which was occurring at a much slower rate. Sinking due to eustacy was computed to be 9 cm between 1908 to 1980; in this same period natural subsidence, partly due to sediment compaction, was determined to be 3 cm (Gatto and Carbognin, 1981). These figures for subsidence are not surprising since the existence of high seismic risk areas and thermal zones not far from Venice is widely known. Consequently orogenic, tectonic phenomena and perhaps the effects of campaction due to the cooling of the lithosphere could not be excluded. It results then, that in the actual condition of increased precariousness, even assuming that the man-induced factor connected to the exploitation of groundwater has been completely eliminated, the natural phenomena should be studied. As a consequence, a programming of geodetic controls by traditional measures along opportune reference lines and, on a long term basis, by remote sensing techniques (satellite geodetics) seems appropriate.

The Venice Situation After 1975

In spite of the optimistic expectations consequent to the ceasement of subsidence, the hydraulic head and ground surface elevation of the lagoon area were equally kept under strict control: land stability can in fact be upset not only by human intervention but also by natural causes which are difficult to predict.

A network of 10 piezometers placed in the more critical points of the Venetian area, and opportune field measurements have given a continuous monitoring of the hydraulic head which up to 1977 has continued to increase at a considerable rate (Fig. 1). The most significant recoveries were recorded at the, one time, very subsiding zones (Fig. 2); only in recently industrialized areas of the hinterland are static or local drawdown conditions noted.

On a whole therefore, for the territory closest to the sea and more exposed to the dangers of flooding, the hydrological situation is reassuring and does not present any problem.

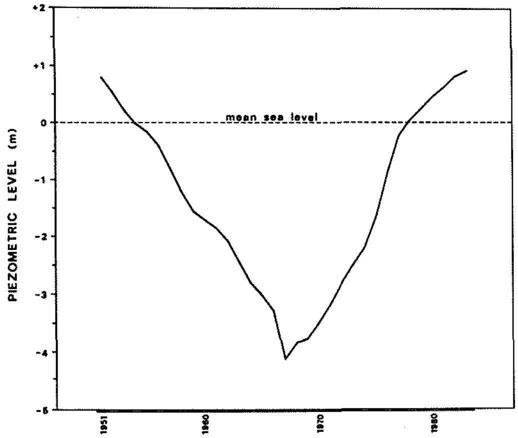

Fig. 1. Piezometric levels of the 120 meter-deep "Rialto Nuovo" well in Venice. A reverse trend in the upper aquifers was already noted in 1967 when exploitation of these was decreased and water was supplied to the islands and the littorals by the city aqueduct.

Instead, the elevation of the city seems to be quite different. A leveling conducted in 1977 by the Direzione Generale del Catasto e dei Servizi Tecnici Erariali to control vertical movements of the Apennines, Alps and the Po Valley, would indicate , besides changes in elevation of some bench marks located in the hinterland, even a reactivation of subsidence in Venice. According to these levelings, which run along the survey line starting at Pennabilli (A. Umbro), passing through Venice and finishing at Ponte nelle Alpi, a land sinking of almost 3.5 cm in the lagoon area, with respect to the levelings made by CNR in 1975, which, if continually occurring, would indicate a rate of 1.7 cm/y, detrimental to the survival of Venice. A new survey carried out in 1980 by the Ufficio Idrografico del Magistrato alle Acque (Ministero dei Lavori Pubblici) seems to confirm a sinking with respect to the 1975 leveling.

The results from the last levelings, all carried out according to specifications for high accuracy, conflicting with the evolution of the piezometric values, prompted further, extensive investigations. The existence of this sinking occurrence would come to be even disproved by the considerations of the tide gauges. The comparison between the recorded mean sea levels at Venice and those of a few tide stations along the Yugoslavian coast (Rovigno and Buccari), would confirm the land stability even after 1975 (Fig. 3). It seems thus reasonable to suspect that the apparent recurrence of the recorded subsidence between 1975 and 1977 is rather attributed to a rising of the basic reference for geodetic measures located at the foot of the hill area affected by a seismogenetic structure which was shown to be active in 1976. The importance that land stability has on the effectiveness of the hydraulic works projected to defend Venice has prompted the establishment of a new first order leveling which, starting from the stabler, tectonic dolomite zone, checks elevations on the basis of the reference of preceding measurements and of Venice.

New Geodetic Control Techniques

The 1961 leveling, the first to show land sinking in the Venetian area, started at the base of Conegliano, at that time considered tectonically

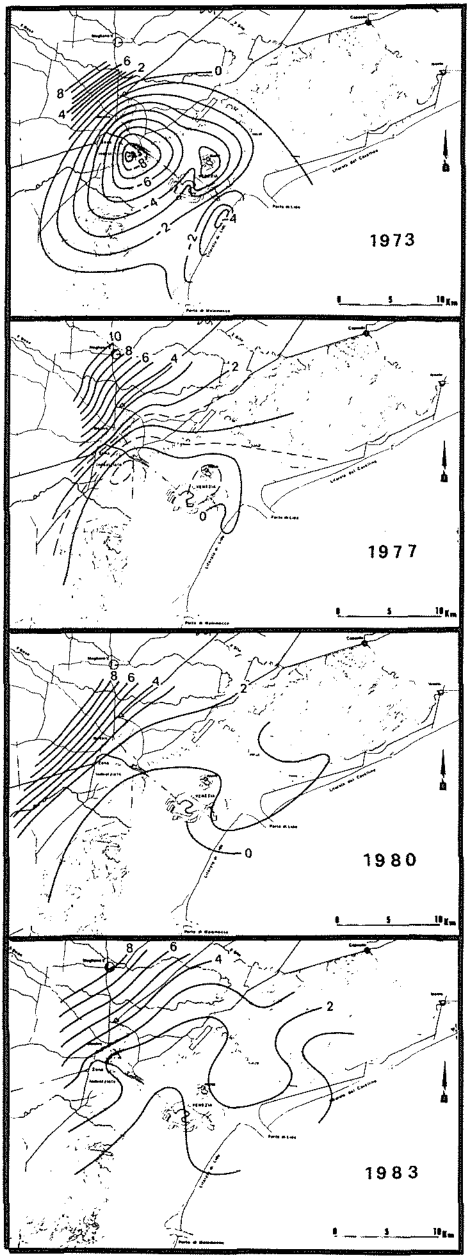

Fig. 2. Piezometric evolution of the 4th aquifer in Venice. Equidistance 1 m (1973: after Mozzi et al., 1975; 1977- 80: after Carbognin et al., 1981.

stable. Successive levelings, even if made by different agencies, always referred, directly or indirectly, to the elevation of this base (horizontal bench mark, I.G.M. 35/49). Only in 1975 did CNR, concerned that the reference for levelings to check changes in elevation of Venice was a single bench mark established in loose alluvial soils, place several new bench marks embedded in rock at Rua di Feletto not far from the Conegliano one.

The earthquake of 1976, which as it is known, originated from a seismogenetic structure involving the region north of Conegliano, upsetting the elevation of the Friulano-Veneto plain, causing extensive vertical movements. The 1977 leveling, made by the IGM, showed in fact risings all over the Friuli plain with maximum values (24 cm) corresponding to the mentioned tectonic discontinuity and minima towards the coast. From the trend of equal lines of subsidence (Fig. 4), the importance of such movements, which weaken as they move towards the Veneto plain, seems evident. It is therefore probable that the bases at Conegliano and Rua di Feletto have undergone a certain rising. With regards to this, it is important to point out a wide movement of the land shown by the 1975 levelings (Fig. 5), which affected the area for about 20 km to the south of a tectonic discontinuity located in correspondence with Ponte della Priula (Conegliano-Treviso line). It is an arching of the land which reached a maximum height of 1.8 cm; this heaving, no longer present in 1977, was explained as the effect of the tectonic "pushing" from the south towards the north. Exhausting the strong, active stresses through the earthquake, the ground would assume its initial elevation.

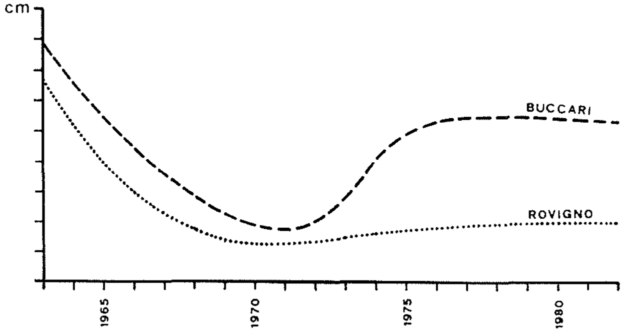

Fig. 3. Ground elevation differences at Venice estimated from recorded mean sea level differences at Venice with respect to those recorded at Rovigno and Buccari. The occurence of subsidence in the 1960's and subsequent ceasement of it are evident.

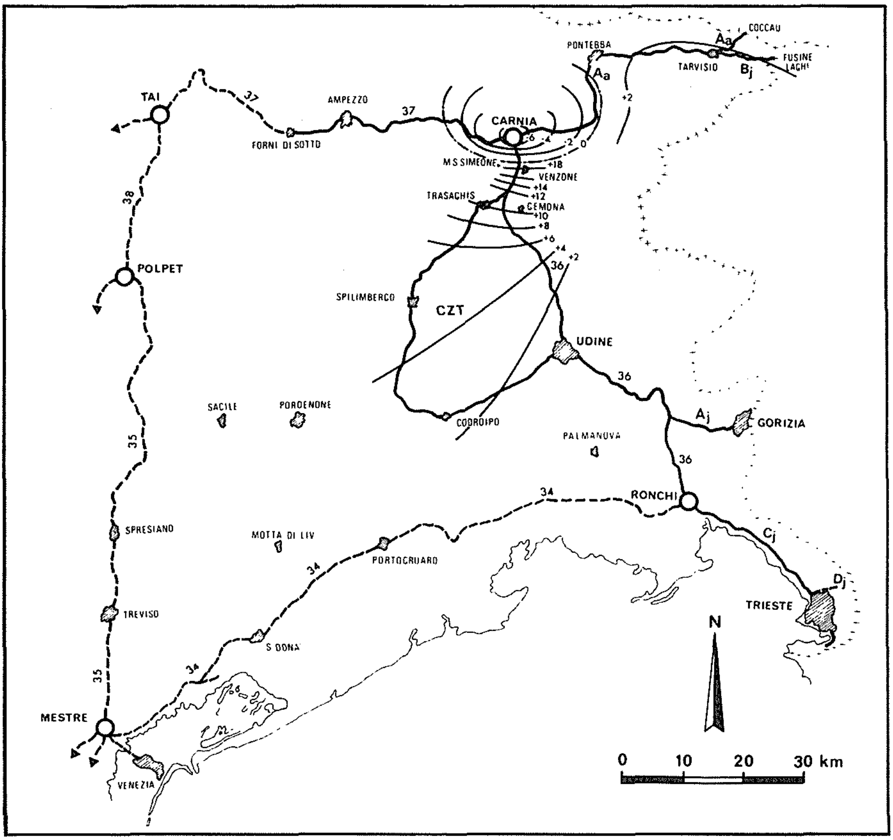

Fig. 4. Ground elevation changes in the areas affected by the 1976 earthquake (after Talamo et al., 1978).

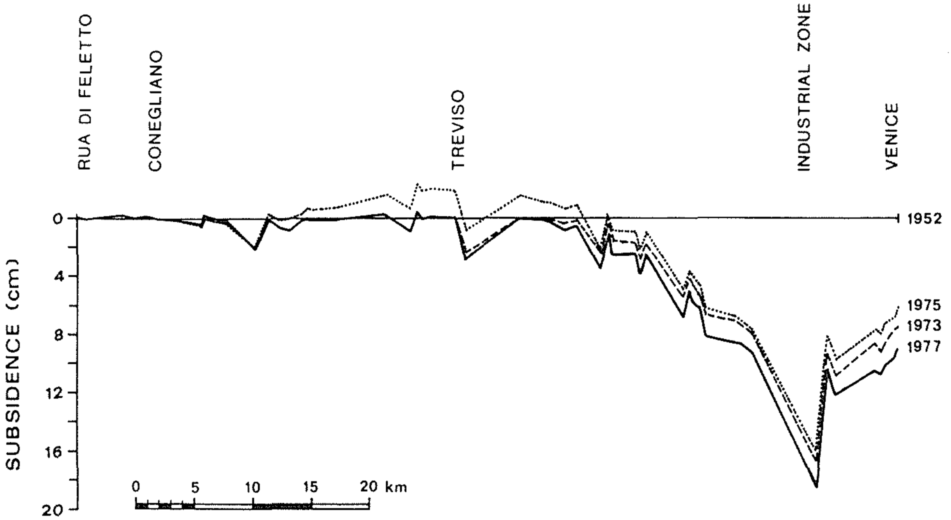

Fig. 5. Comparison of levelings between 1973 and 1977.

Since the Veneto layers at the foot of the hills is not stable, it is necessary to rely on a new geodetic survey starting from the Dolomite area, which is located outside the principle active tectonic structures of Veneto (Valsugana fault, Periadriatica folded fault, etc.). The new levelings, which will cover over 350 km, will begin at Cortina d'Ampezzo and will follow two distinct routes (Fig. 6). The first to be surveyed by the Veneto Region will run along the IGM level lines 39 (Cortina d'Ampezzo-Tai di Cadore), 38 (Tai di Cadore-Polpet) and 35 (Polpet-Mestre). The U.I.M.A. will survey the second levelings whose bench marks should be materialized for a good part again ("ex novo"). This level line will follow the Cordevole, Piave and Brenta valleys pass through Castelfranco, and will reach the nodal point at Mestre. The two first order level lines will be opportunely linked to each other, so as to form three closed polygons, which will allow a more correct compensation for elevations surveyed. From the nodal at Mestre, the leveling line will follow the CDV3 line to Venice and will end with the bench mark placed at the "Punta della Salute", representative of elevation movements in Venice and near to the "Punta della Salute" tide gauge.

To be significant, the level line will be surveyed in the shortest time possible with the cooperation of a number of surveying groups and it must not exceed small discrepancies. The leveling, initiated in December 1983, was interrupted due to unexpected intense cold weather, which hindered the field work, and will be resumed in the spring of 1984. The final results will be published together with an extensive bibliography on completion of the survey.

Conclusions

This geodetic survey is included as one of the controls which local authorities promote to warn of new elevation lowerings, which would be a catastrophe for the survival of Venice and its lagoon. It will permit above all to

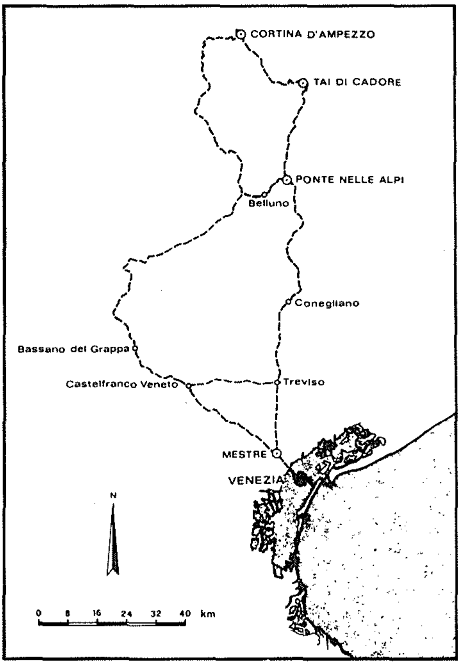

Fig. 6. New survey net.

*verify the actual ground level and to confirm the causes of recent recur*rences, in frequency and height, of the phenomenon of flooding.

By the determination of a precise, updated elevation, reference data will provide a new surface elevation for Venice and the surrounding area which is of upmost importance in urban renewal works and in actualizing the flood control projects planned by the Ministry of Public Works.

Geodetic leveling lines newly established and those already existing, which cross the whole territory, constitute a reference base for all altimetric determinations for territorial, technical uses. The survey will even allow to demonstrate the effects of active, neotectonic structures in Venice by comparing geodetic data previous to and following the 1976 earthquake. Finally, these levelings are a part of the investigation of the natural elevation trends of Venice.

Acknowledgements

The authors wish to thank Prof. A. Tomasin for supplying the tidal data and Mr. G. Zambon for the graphical work.

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GEOTECHNICAL PROPERTIES OF MODENA SUBSOIL: A PRELIMINARY REPORT

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Abstract

Results of field and laboratory geotechnical investigations, carried out in the territory of Modena (Northern Italy) that is presently affected by intensive subsidence phenomena, are reported. The subsoil sequence is formed by alluvial silty clays, with interbedded lenses of sandy gravels; friction-cone penetration tests (CPT) are currently adopted for routine soil investigations. The upper part of clay deposits and some deeper horizons were overconsolidated; however, most of clay layers appear normally consolidated and their compressibility is responsible for the measured amount of subsidence. The available data about properties of Modena soils are reported and discussed; finally, a program for further, deepened soil investigations is put forward.

Introduction

The town of Modena is situated in the southern part of the River Po Basin (Northern Italy), wide sectors of which are presently affected by intensive subsidence phenomena. During the last twenty years a total settlement close to one meter was attained, and several important, monumental and historical, buildings, within the center of the town, suffered severe damages due to differential settlement of foundations.

In the sphere of topographical, hydrogeological, civil engineering and architectural studies, concerning the multifarious aspects of the ground lowering, geotechnical investigations are planned, the aim of which should be a, detailed and deepened, geotechnical model of the portion of subsoil that is subject to the subsidence phenomenon. The till now available data allow us a first glance to the soil properties; meanwhile, ad hoc investigations are in progress and will form the object of successive studies.

Geological Setting

The area belongs to the basins of rightside tributaries of the River Po, descending from the Apennines, at the end of the alluvial fan of River Secchia and about 15 km far from the Apennine border.

The township area extends over recent alluvial deposits (Medium to Upper Pleistocene and Holocene), more than 300 m thick, overlying marine clays (Lower and Medium Pleistocene). The alluvial sequence is formed by clays and silts, with interbedded lenses of sandy gravel, the thickness of which tends to decrease from southwest to northeast (Colombetti et al., 1980). In the examined area, the upper gravel layer is about 20 m deep and tends to deepen from SW to NE; other gravel lenses are found at greater depths.

The typical subsoil profile is represented by the deepest (till now) geotechnical boring, that is positioned in the town center (Piazza Grande, in front of the Town-Hall); two gravel layers have been crossed by the borehole within the first 68 m (see fig. 1a).

The alluvial sequence is the seat of a monolayer aquifer system, bearing confined groundwater. The piezometric level was prevailing till the end of the last war, but presently it is about 7 m deep in the town center. Likely, the land subsidence of the last twenty years (up to about 1 m) can be imputed to lowering of the piezometric level, by 8 ÷ 10 m since 1945 up today (Colombetti et al., 1984; Cancelli & Pellegrini, 1984).

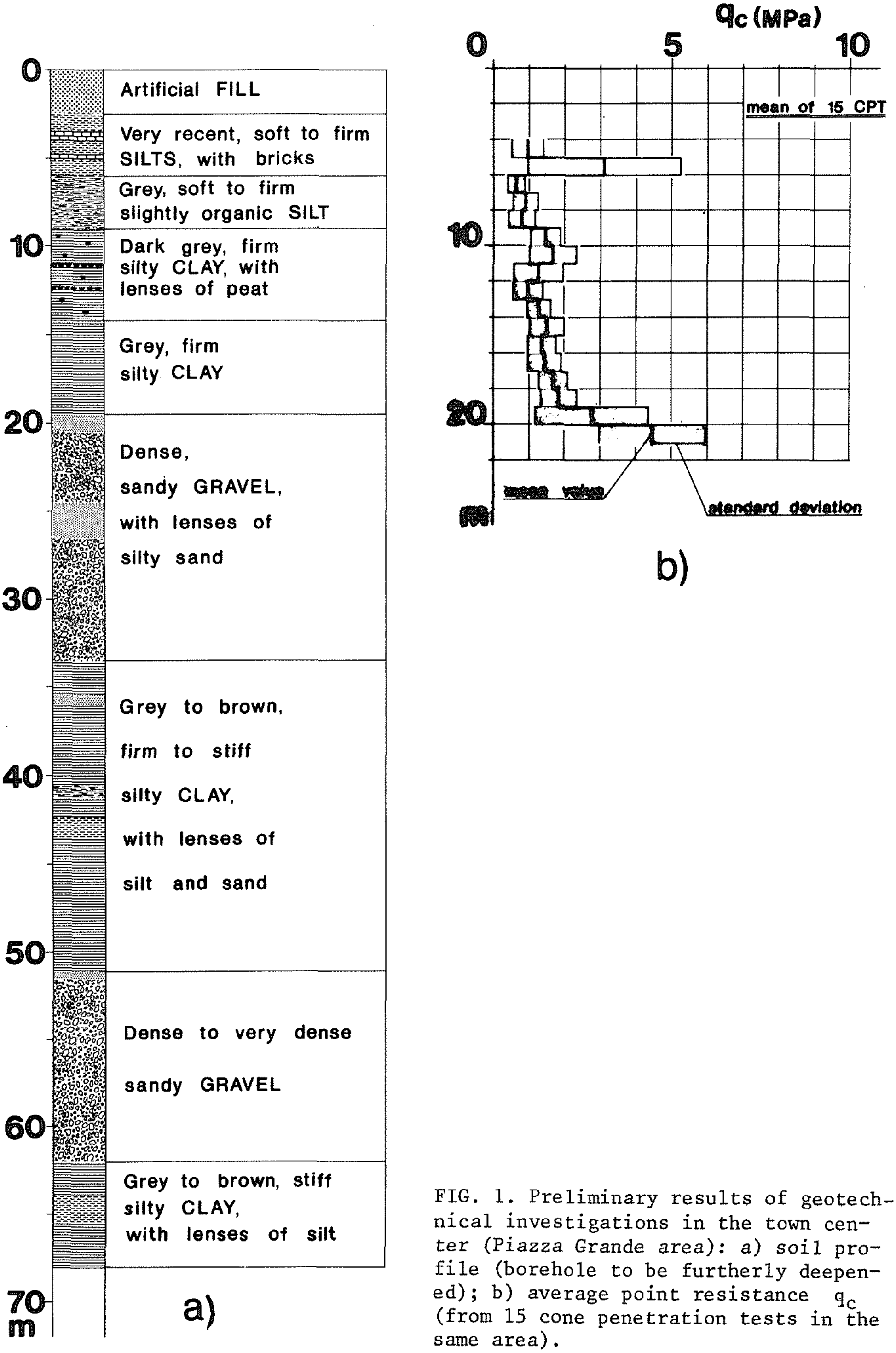

FIG. 1. Preliminary results of geotechnical investigations in the town center (Piazza Grande area): a) soil profile (borehole to be furtherly deepened); b) average point resistance q_c (from 15 cone penetration tests in the same area).

Also from fig. 1a, it can be observed that the Roman archaeological level (about 2000 years B.P.) is about 6 m deep; therefore, the outcropping soils, formed by silt-clay loam, have a very recent origin.

In Situ Investigations

Apart from a few cases, concerning buildings of outstanding importance, geotechnical investigations are generally limited to the upper silt-clay horizons (20 ÷ 25 m), and are stopped when reaching the first gravel layer.

Only mechanical, quasi-static, friction-cone penetration tests (CPT) are commonly carried out for routine investigations; the use of Begemann's friction mantle cone is widespread. Geotechnical borings, deeper investigations (down to the lower clay strata) and more sophisticated devices are seldom used for important geotechnical problems. Data from laboratory tests on undisturbed soil samples are relatively scarce (Righi, 1980; Martinotti et al., 1981; Cancelli et al., 1982).

Owing to a well established experience, the correspondence between CPT's and borings is generally satisfactory; as an example, the fig. 1b reports the mean values of the point resistance q_c versus depth, as computed by 1 m intervals by taking into account the results of 15 CPT's performed in the town center (Piazza Grande area): the agreement with the stratigraphic profile of fig. 1a is outstanding.

As well known, the friction ratio R_f between the sleeve friction f_s and the point resistance q_c is a useful tool when interpreting the soil profile in the absence of borings (Begemann, 1965; Schmertmann, 1969; Searle, 1979). However, it has to be remembered that the validity of any interpretation charts, based on f_s/q_c relationships, should be restricted only to submerged soil layers, while the common practice doesn't distinguishes between submerged and non-submerged soils. A temptative comparison between the values of q_c and R_f measured for Modena silty clays and a simplified version of Searle's chart (fig. 2), shows that the agreement can be considered satisfactory only for the deepest, fully submerged soil layers (below 10 m).

The point resistance q_c is popularly interpreted in terms of undrained stresses, and the undrained cohesion c_u can be computed by means of the classical bearing capacity formula.

For routine design purposes, a common practice in Modena consists in adopting the simplified formula (see also Heijnen, 1974)

$$c_u = q_c / N_k \qquad (1)$$

in which the overburden pressure is neglected, and values of 13 ÷ 15 are taten for the cone factor N_k.

However, values of N_k much higher than 15 were found for Modena silt-clay soils: a statistical analysis, for which values of c_u determined by UU triaxial tests on undisturbed samples were taken into account, led to average values of N_k as high as 25 (Cancelli et al., 1982). Such results can be justified by the overconsolidation degree of part of Modena soils (in particular within the first 10 m from the ground surface, but OC layers are encountered also to depth, as it is proved by the interbedding of brown, exsiccated horizons into the greyish clay sequence - cf. the soil profile in fig. 1a).

Bearing in mind the depositional path, the observed overconsolidation could have been caused by successive periods of exposure to air, followed by recovering of the exsiccated horizons with new alluvial deposits (Cancelli et al., 1982).

As to the topographical distribution of the main geotechnical design parameters, the following conclusive remarks can be done from in situ investigations:

a) the depth of the upper gravel layer ranges from 12 ÷ 14 m (south-west outskirts) to 18 ÷ 22 m (central part of the town) and to more than 30 m (in

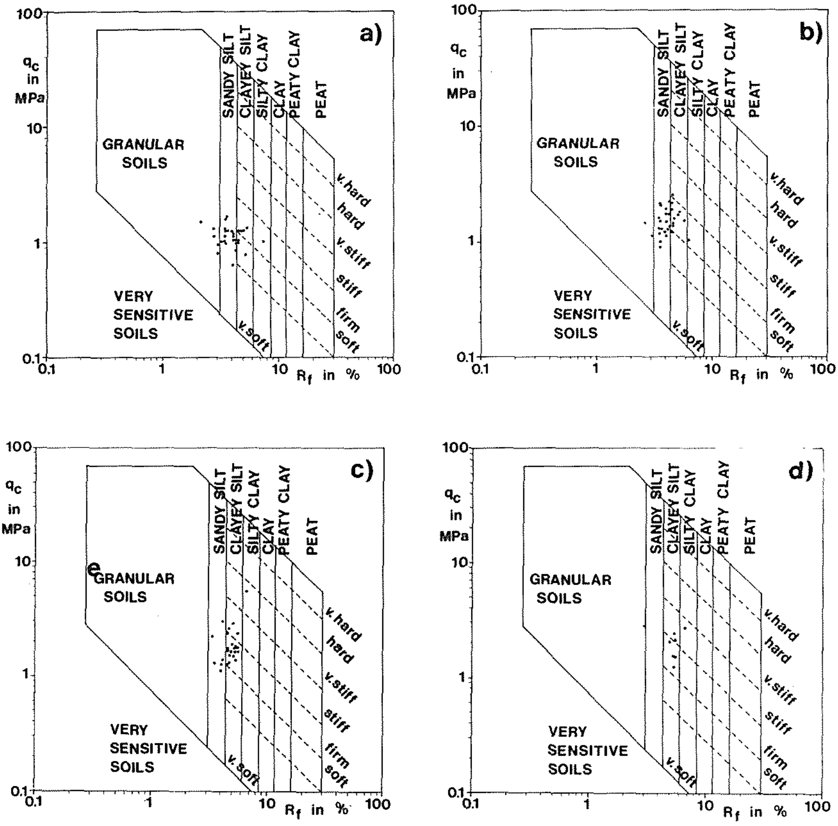

FIG. 2. Interpretation of soil profile by CPT's according to the simplified Searle's chart, by depth intervals: a) 0 ÷ 5 m; b) 5 ÷ 10 m; c) 10 ÷ 15 m; d) 15 ÷ 20 m.

the northern outskirts (Cancelli & Pellegrini, 1984);

b) silt-clay deposits form about 70 % of the upper 120 m of the alluvial sequence, that are likely subject to artificial subsidence by water withdraw, and the relative importance of cohesive layers tends to increase from S to N (Cancelli & Pellegrini, 1984);

c) the average values of q_c in the upper clay layer, computed for 39 groups of CPT's as reported in fig. 1b, range from 0.5 up to 2.5 MPa;

d) the most favourable soil conditions are encountered in the south-western and in the south-eastern outskirts of the town.

Accordingly, the current building practice consists in shallow foundations (mostly on continuous beams), with applied loads ranging from 60 up to 120 kPa; partly floatating mats and, more frequently, deep foundations on cast-in-situ piles, bored to the gravel layer, are employed for the most important and heavy buildings.

Properties of Cohesive Soils

According to the available grain size analyses (limited to the first 20 ÷ 25 m), the composition of cohesive soils varies from silty clays to more or less silty loams, the content in sandy fraction decreasing from surface to depth

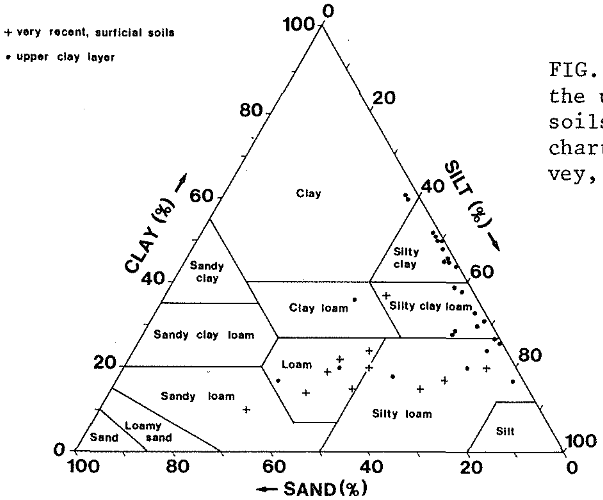

FIG. 3. Grain size composition of the upper 20 ÷ 25 m of cohesive soils (triangular classification chart according to U.S. Soil Survey, USDA, 1960).

(fig. 3) and from S to N; the clay fraction is generally varying from 20 up to more than 60 %.

The plasticity characteristics (liquid limit w_L = 25 ÷ 80 %; plasticity index I_P = 10 ÷ 60 %) are typical of inorganic clays (CL to CH, see plasticity chart in fig. 4). Both parameters seem slightly lower in very recent, surficial soils; however, a unique regression line can be computed, reading

$$I_P = 0.78\ w_L - 13.51\ . \qquad (2)$$

The index of activity is generally close to 0.75, according to a slight prevalence of illite in the clay fraction (Cancelli et al., 1982).

The natural density of silt-clay deposits can be expressed by the following range of the parameters:

- water content w_o = 25 ÷ 55 % (the degree of saturation being very close to the unit, also in the most surficial clay strata);
- unit weight γ = 17 ÷ 20 kN/m^3.

The degree of consistency is highly variable, depending on the stress history (alternance of normally- and of over-consolidated layers). Accordingly,

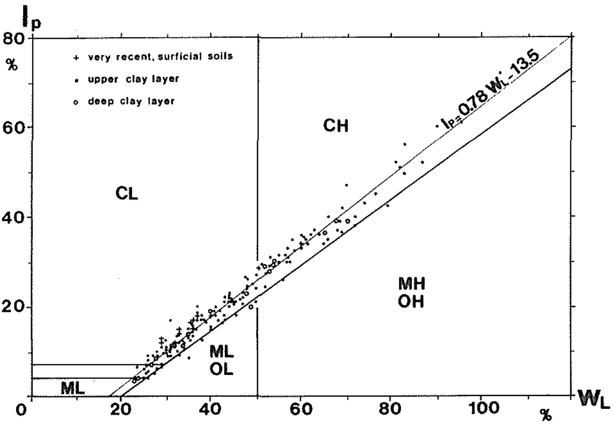

FIG. 4. Plasticity chart.

the undrained cohesion c_u, as determined by laboratory, UU triaxial or unconfined compression, tests, ranges from 30 to 100 kPa; most of the highest values cluster within the first 10 m from the ground surface, and can be likely correlated to the presence of, overconsolidated by surface desiccation, clay horizons; the lowest values fall between 10 and 15 m of depth, in accordance with the existence of underconsolidated clay layers (see after).

Several results of conventional, increment load, oedometer tests are available for Modena clay soils, both unpublished and published. From void ratio versus log of the effective pressure plot, the maximum apparent preconsolidation pressure p_c' has been evaluated by Casagrande's method. The range of probable values of p_c' is reported versus depth in fig. 5, and compared to the geostatic pressure p_o' at different times (respectively, before and after the lowering of the groundwater piezometric level); two typical areas of Modena territory are represented, namely:

a) the central part of the town, where the piezometric level lowered by 8 ÷ 10 m;

b) the northern outskirts, close to the River Secchia, where the lowering was practically negligible.

Overconsolidated layers (with overconsolidation ratio OCR ranging from 2 to 4) are present at various depths, and mostly in the first 10 m from the ground surface; moreover, slightly overconsolidated layers (with OCR ≤ 2) are erratically found also at major depths, in accordance with the presence of reddish and brown interbeddings in the soil sequence.

It is important to note that in the central area (fig. 5a) some samples, 10 ÷ 15 m deep, appear underconsolidated: this fact is the prove that a land subsidence phenomenon, related to the recent lowering of the groundwater piezometric level, is still ahead. On the contrary, no underconsolidated layers were found in the northern outskirts (fig. 5b).

Apart from over- or under-consolidated interbeddings, the prevailing part of deep silt-clay soils is normally consolidated (at least at present).

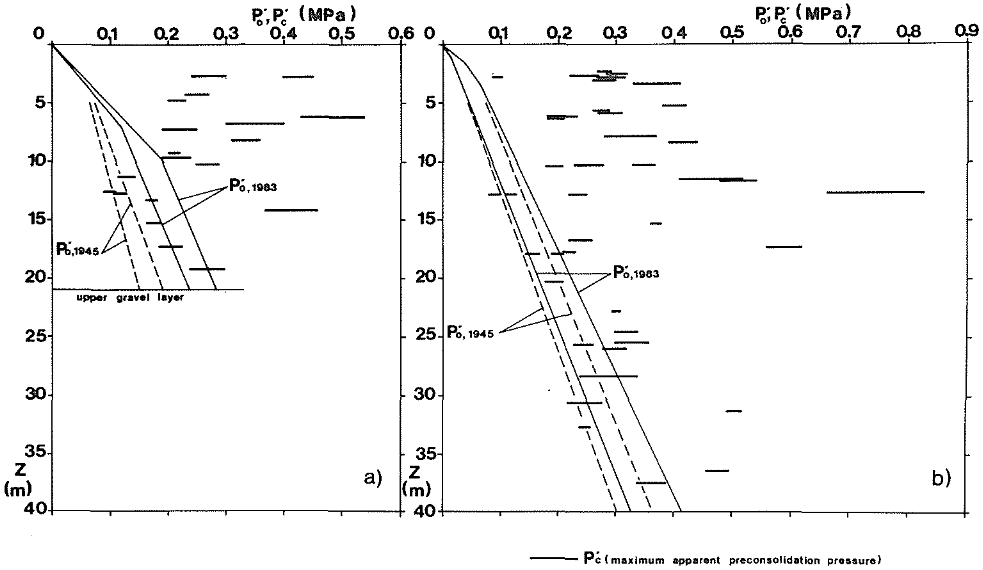

Fig. 5. Apparent preconsolidation pressure p_c' and geostatic pressure p_o' (at different times), vs. depth: a) central part of the town; b) northern outskirts.

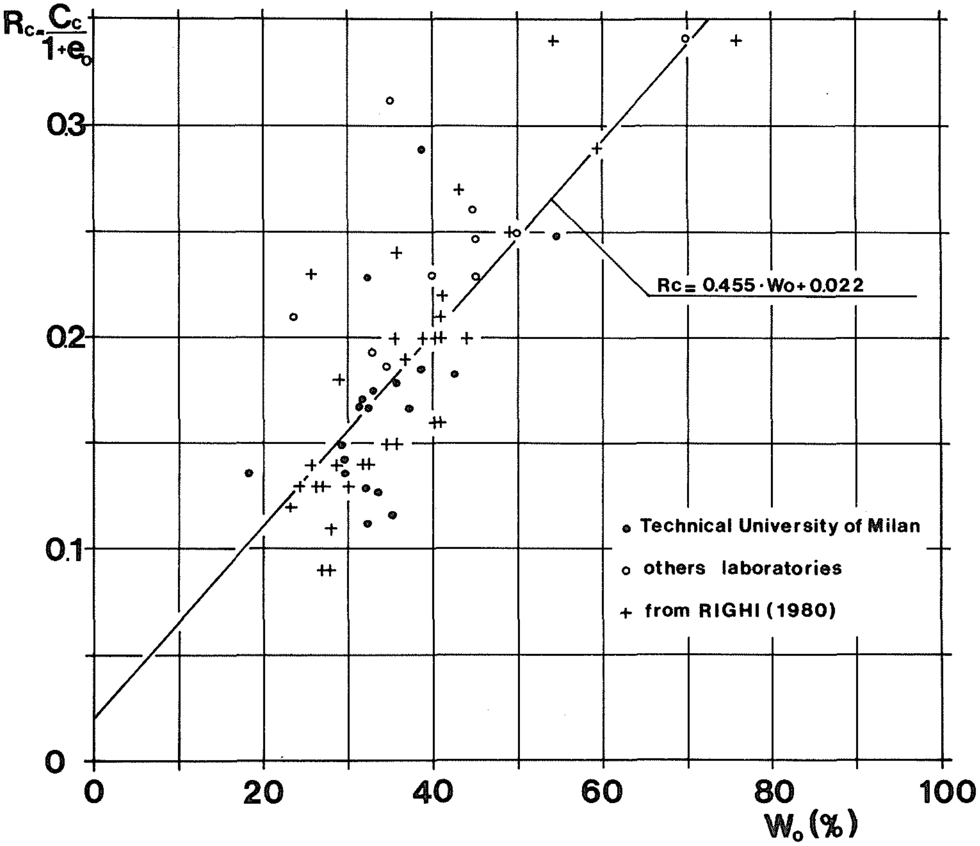

FIG. 6. Virgin compression ratio R_c versus natural water content w_o.

The primary (consolidation) compressibility of normally consolidated soils can be represented by the virgin compression index $C_c = \Delta e / \Delta \log p'$, or, more practically, by the virgin compression ratio $R_c = C_c / (1 + e_o)$, in which e_o is the initial void ratio. Values of R_c ranging from 0.1 to about 0.3 were obtained, and a representative average value of $R_c = 0.2$ can be assumed for Modena soils; besides, a linear correlation between R_c and the natural water content w_o can be established, that is

$$R_c = 0.455\ w_o + 0.022 \qquad (3)$$

with a coefficient of linear correlation r = 0.74 (see fig. 6).

As to the coefficient of vertical consolidation, values of $c_v = (0.5 \div 2) . 10^{-7} m^2/s$ were found, that are consistent with the liquidity limit w_L of the investigated soils (Navdocks, 1962). Such values, together with the lack of permeable interbeddings within the main clay layers, give reason for the low rate of consolidation settlement following the lowering of the groundwater level, and for the existence of underconsolidated layers.

Finally, several oedometer tests were devoted to investigate secondary compression, as a possible factor of geological subsidence. For normally consolidated samples of Modena clays, the secondary compression ratio $C_\alpha = \Delta \varepsilon / \Delta \log t$ ranges from 0.5 to 1.1 %; the ratio $C_\alpha / R_c = 4 \div 5$ % can be considered as relatively low in comparison with other natural soils and, consequently, the secondary time effect can be practically disregarded.

In order to complete the outlines of geotechnical properties of cohesive soils, the effective shear strength parameters can be drawn from results of CID and CIU triaxial tests, as reported in fig. 7. The curved upper envelope is consistent with the overconsolidation of part of clay deposits; the range of values of the effective angle ϕ' ($17 \div 24$ °) is slightly lower than the range of values that one should expect on the basis of plasticity index.

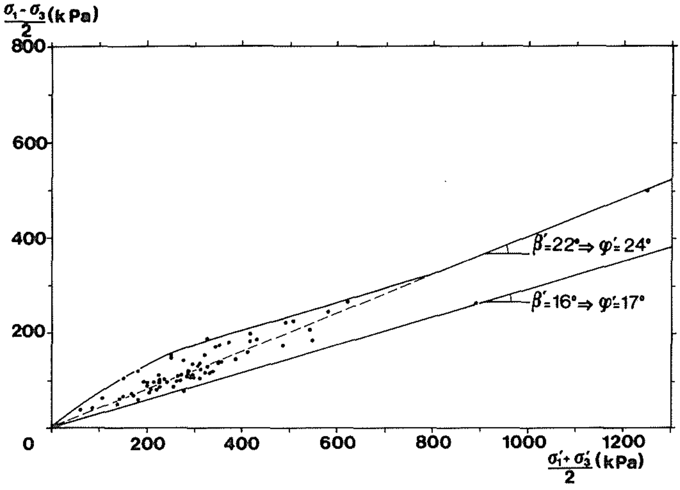

FIG. 7. Effective stress envelope from triaxial tests.

Properties of Granular Soils

As previously said, the upper coarse grained layer lies at depth increasing from South to North and varying from 18 to 22 m in the central part of the town. If lenses of silty sand at the top and within the stratum (see soil profile in fig. 1a) are included, the overall thickness varies from 5 up to about 14 m.

The main lithotype is formed by sandy gravel or by gravel with silty sand; the cobbles are always well rounded and their maximum diameter is of the order of $50 \div 100$ mm.

In order the relative density of gravelly soils be evaluated, standard penetration tests are currently performed into boreholes. High values of the dynamic resistance N_{SPT} are generally measured: a mean value $\bar{N}_{SPT} = 50$ and a standard deviation $s_N = 11$ blows/foot can be computed; the lowest values of N_{SPT} correspond to silty sand lenses.

According to well established, routine interpretative criteria, such values of N_{SPT} imply:

- a relative density $D_r \geq 0.8$;
- an angle of shearing strength $\phi' \geq 38 \div 40$ ° ;
- an "oedometric" modulus $E_{oed} \geq 50$ MPa.

Consequently, the gravel layer constitues a good basement for most piled foundations and its compressibility can be neglected, in comparison with the compressibility of silt-clay layers, when considering a general land subsidence.

Summing Up Present Knowledge

On the basis of the up today available data, the following statements have to be pointed out:

a) Modena subsoil is formed in prevalence (60 ÷ 80 % on the total) by alluvial, silt-clay deposits, with interbedded layers and lenses of sandy gravel (bearing artesian water);

b) the upper 10 m and some other deeper layers of cohesive soils appear to have been overconsolidated by surface desiccation due to a past exposure to air; however, the main part (about 50 ÷ 60 % on the total alluvial sequence) seems normally consolidated;

c) by oedometer tests, some underconsolidated soil layers were recognized, as a prove of consolidation subsidence process, not yet exhausted;

d) the subsidence phenomenon can be imputed to the lowering of groundwater piezometric level since 1945 up today (about 10 m in the town center, corresponding to a vertical stress increment of about 0.1 MPa);

e) a mean value of the compression ratio equal to 0.2 can be assumed for a first crude estimate of consolidation settlement in cohesive soils;

f) gravelly soils possess high density and their compressibility can be neglected in comparison with the compressibility of cohesive soils.

The present geotechnical knowledge of Modena subsoil is based upon data made available from routine investigations, that are generally limited to the upper 20 ÷ 30 m. As the stress increment should be applied to the whole aquifer layer subject to water withdraw (about 120 m thick), further, detailed and deepened soil investigations are needed, in order to achieve a better understanding of land subsidence phenomena.

Future Investigations

By care of the Municipality of Modena, a special program of soil investigations has been formulated for the town center, and especially for the area of Piazza Grande, that is surrounded by several historic monuments (as the Cathedral, the Ghirlandina Tower and the Town-Hall).

New deep borings, down to 80 ÷ 100 m, are in progress; the geotechnical instrumentation to be installed will include:

- hydraulic piezometers (Casagrande type) into aquifer, gravel lenses;
- electropneumatic piezometers at different depths into clay layers under consolidation;
- magnetic settlement gauges at different depths.

Finally, to a better definition of soil properties, proper laboratory tests on undisturbed soil samples are provided.

Acknowledgements

The present research work and the presentation of results to poster session have been supported by the Municipality of Modena.

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ANALYSIS OF DIFFERENTIAL SETTLEMENTS ON MONUMENTAL STRUCTURES BY MEANS OF THE "DAG" AUTOMATIC MEASURING DEVICE OF LEVELS AND INCLINATIONS

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Abstract

Automatic Measuring Device of Levels and Inclinations.
System for long-term continuous measurements of vertical level variations of any number of locations with reference to one of the locations. Here are reported the System's main features and its applications on monumental structures.

Introduction

The DAG System, of SIS Geotecnica-CISE patent, has been designed and developed at CISE as part of its research activity to develop new technologies aimed at monitoring the behaviour of big turbines, buildings and monumental structures.

The DAG (Automatic Measuring Device of Levels and Inclinations) is a system apt to measure the level variations of different numerous locations. The measurement is differential in comparison with one of the locations. The peculiarity of this system is its accuracy: 10^{-2} mm among locations set at any distance and even not easily accessible. Measurements are not affected by vibrations or temparature variations.

The DAG System can be largely used whenever a continuous and accurate measurement of big structures - i.e. bridges, dams, buildings, monuments - is needed.

A typical application may be the monitoring of big foundations in difficult soils where stability problems may arise, or structures - such as nuclear power plants - where maximum reliability is required. Additional applications may be: monitoring of settlements in vessels; control of inclinations occurring in tall buildings (monuments, telephone towers, skyscrapers, etc.), control of areas subjected to soil slow motion such as land subsidence and uplift.

Description of the System and its operating principle

The System, based on the principle of the communicating vessels, is made of an hydraulic circuit (fig. 1) and a number of vessels - one for each measurement location - partially filled with a special fluid. All vessels are communicating through both the liquid and the air side. Connection is achieved by a rectangular-section, horizontal pipe partially filled with the same fluid. The horizontal pipe is connected to each vessel by two small tubes: one for the liquid (bottom) and one for the air (top). Each vessel houses two transducers: one for temperature and one for level measurement.

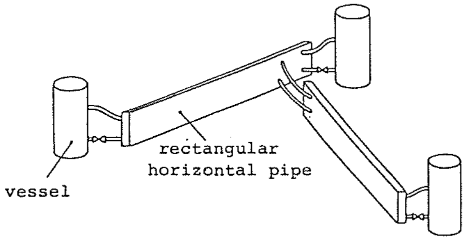

Fig. 1 Sketch of the hydraulic circuit

The DAG System is available in two different models depending on how the measurement of the fluid level inside the vessels is taken.

In the first model (Fig. 2) a proximity sensor is used. The sensor faces a steel float, centered by four small chains, which acts as a target for the sensor and reduces friction down to negligeable values. The transducer, fixed to the cover of the vessel, measures the distance between the float and the cover. The vessel houses the electronic circuit for signal conditioning which allows output of two signals: a level signal released from the thermal effect of the fluid and calibrated at 1V/mm, and a temperature signal. The main feature of this type of DAG System is its high resolution. In fact it may be specifically used wherever high accuracy is required and when level variations are very slight (the System's linearity range is ± 2.5 mm).

In the second model (Fig. 3) the fluid level is measured by an angular transducer, coaxially mounted on a pulley by means of ball bearings. The angular position of the pulley is determined by the fluid level inside the vessel by means of a float with counterweight. The vessel houses the electronic circuit

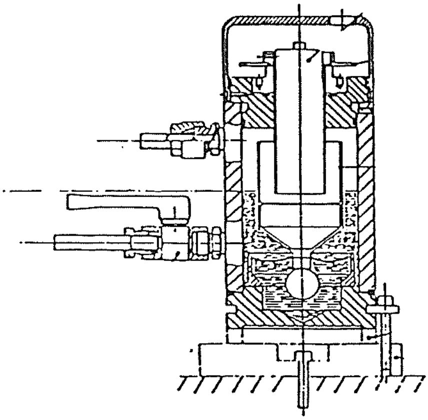

Fig. 2 DAG 10/S type

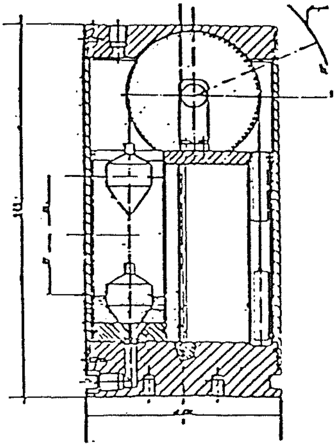

Fig. 3 DAG 100/S type

Cross section of a vessel

"LAST SUPPER"

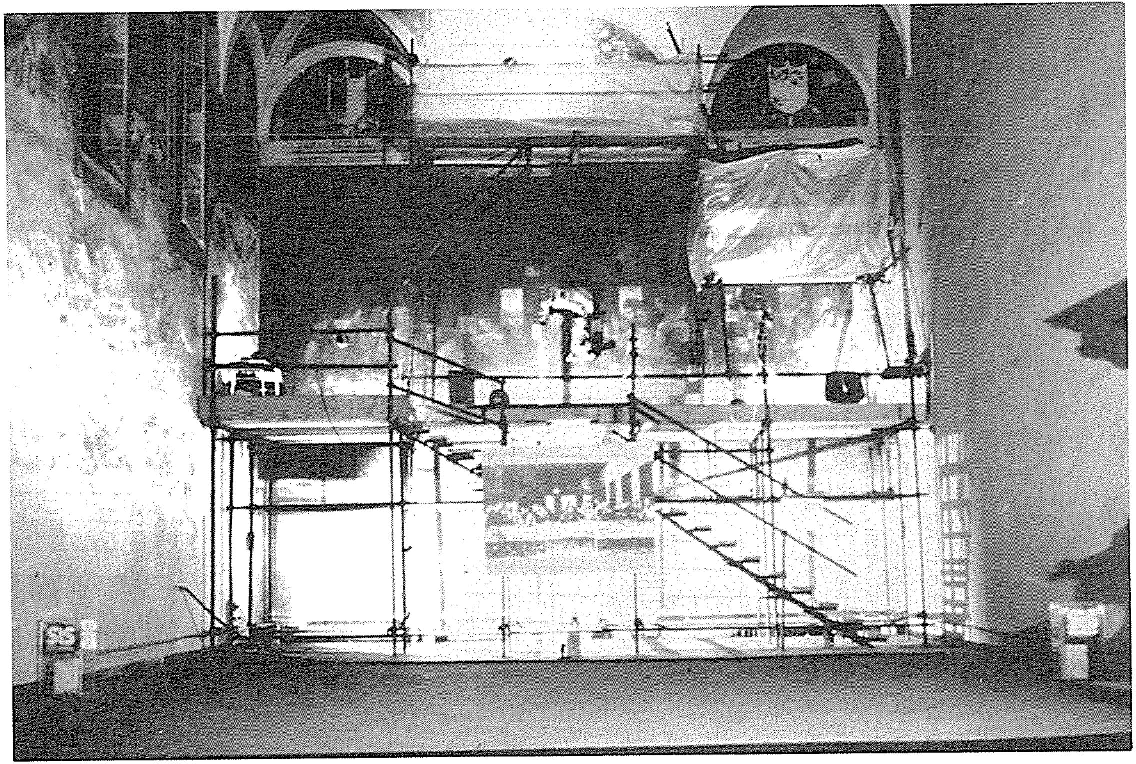

Fig. 4 View of the fresco with two DAG sensors at both sides

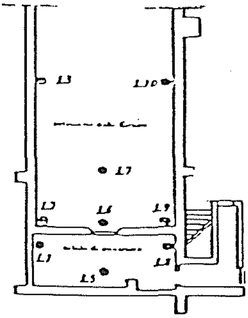

Fig. 5 Plan of the Church with the location of the DAG sensors

for signal conditioning which allows output of two signals: a level signal released from the thermal effect of the fluid and calibrated at 10 mV/mm, and a temperature signal. The linear range of this System is ± 50 mm: thus it suits all those applications requiring high level variations but less high sensitivity.

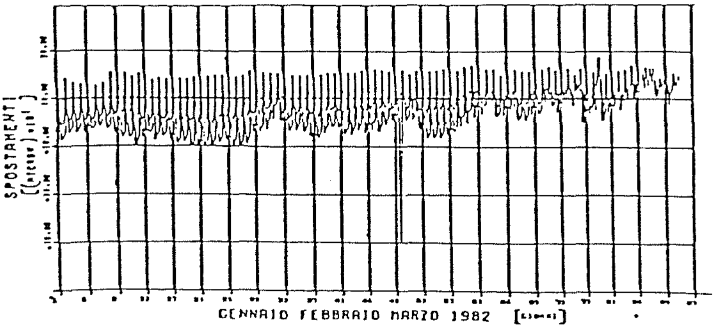

Fig. 6 Measurements of vertical and differential displacements

<u>Outline of the System uses and potential applications</u>

DAG Systems have so far been installed on 6 turboalternators and will soon be installed on two 320 MW units during construction.

In addition DAG Systems have been employed to monitor the behaviour of monumental structures such as: Palazzo dell'Aren-

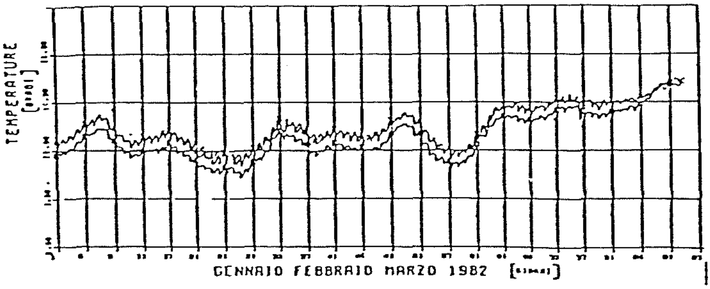

Fig. 7 Measurements of temparature

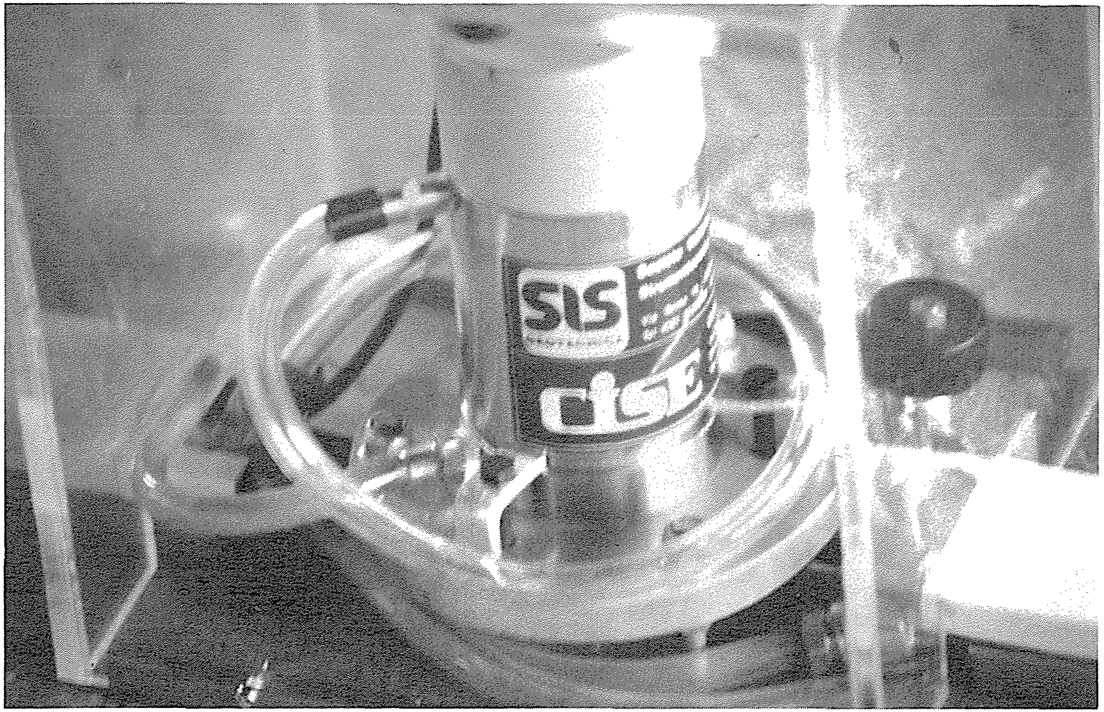

Fig. 8 DAG 10/S type

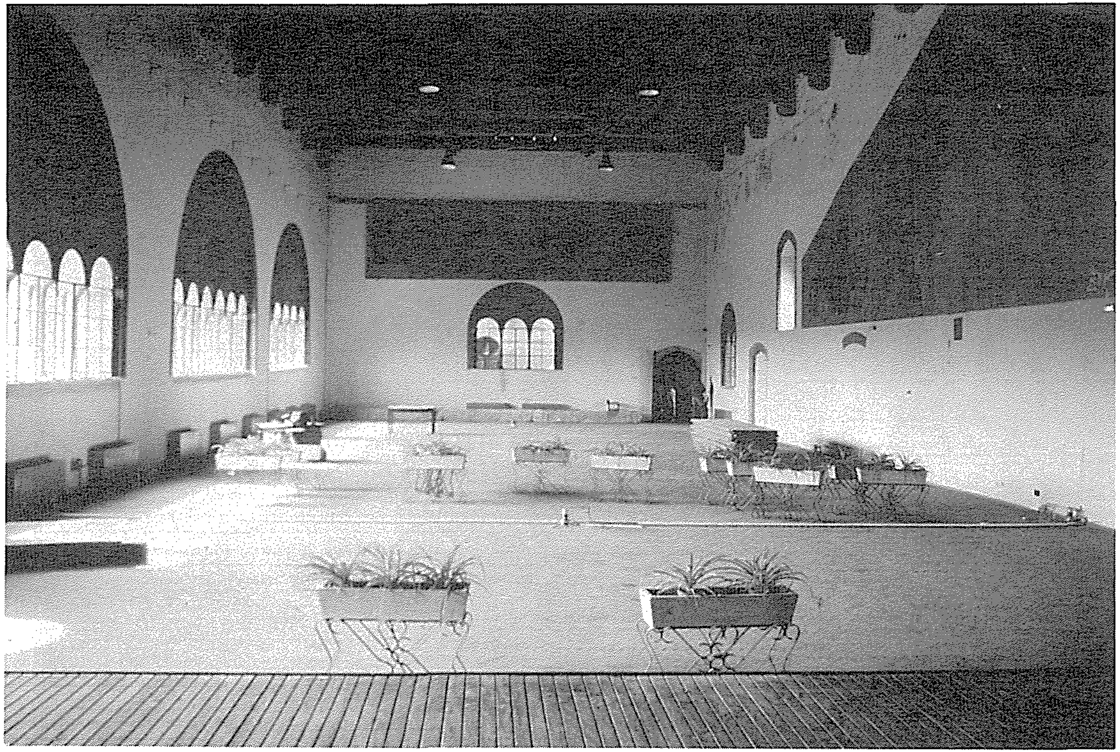

Fig. 9 Installation of a DAG network in the Palazzo dell'Arengo in Rimini

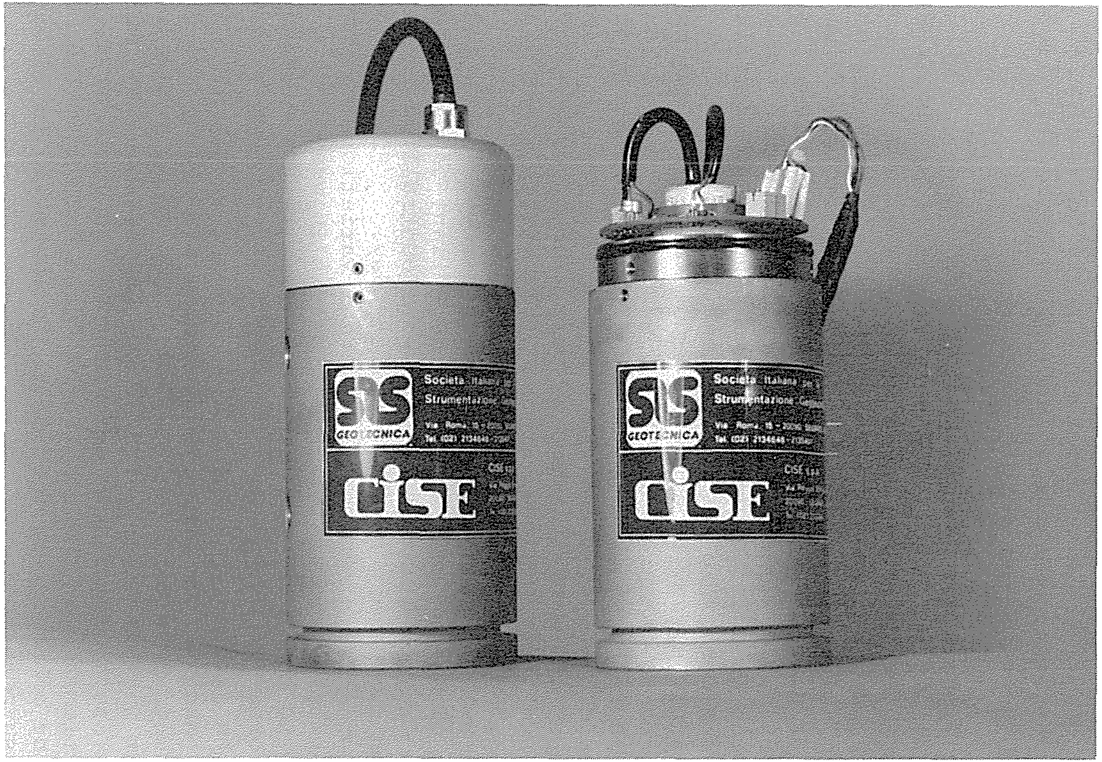

Fig. 10 Automatic Measuring Device of Levels and Inclinations DAG 10/S type

go (Rimini), Church of Santa Maria delle Grazie (Milan) housing the "Last Supper" by Leonardo, Basilica of San Vitale and Mausoleum of Galla Placidia (Ravenna), bell-tower of San Crispino (Rovigo); and of engineering structures such as Italsider rolling-mill (Bagnoli), Dams of Campolattaro (Benevento) and Pontebarca (Sicily).

A new settlement control system for foundation structures of nuclear power plants by means of a fully automated DAG System has recently been developed and is now being installed in a nuclear power plant.

Significan applications on monumental structures

Both the ancient buildings of the Palazzo dell'Arengo in Rimini and the Church of Santa Maria delle Grazie in Milan showed problems of deformations, cracks in the walls, main inside walls out-of-plumb, thus requiring the installation of numerous types of sensors such as direct and inverted plumb lines, pressure cells, joint meters, thermoresistors, water table control systems, in addition to the above mentioned automatic measuring devices of levels and inclinations.

Nowadays the survey of ancient structures has become much easier thanks to the availability of new sophisticated instruments that shorten response time thus allowing prompt intervention when required.

The use of the same instruments will also allow a thorough control of the effectiveness of the intervention works (i.e. injections).

Figs. 4-5-6-7-8 show details of a DAG network installed in the Church of Santa Maria delle Grazie (Milan) on the two walls siding the fresco. All measurements were centralized in one acquisition system and recorded twice a day.

All recorded data showed that thermal variations had produced remarkable deformations. As a result it was of vital importance to equip all sensors with a temperature measurement system in order to detect to which exact extent temperature affected the structure.

Figs. 9 and 10 show some DAG Systems installed in the Palazzo dell'Arengo in Rimini. This system, made up of 8 vessels, showed that in this case as well daily and seasonal thermal variations caused remarkable deformations and cyclical differential settlements on the structure. These data had also been confirmed by other types of instruments installed in 50 different measurement points.

A new instrument, specifically designed to measure horizontal differential movements was also installed in this structure since some of its walls showed remarkable swellings.

A DAG System has recently been installed in the monumental area of Ravenna affected by a serious problem of vast proportions of land subsidence. Around the Basilica of San Vitale

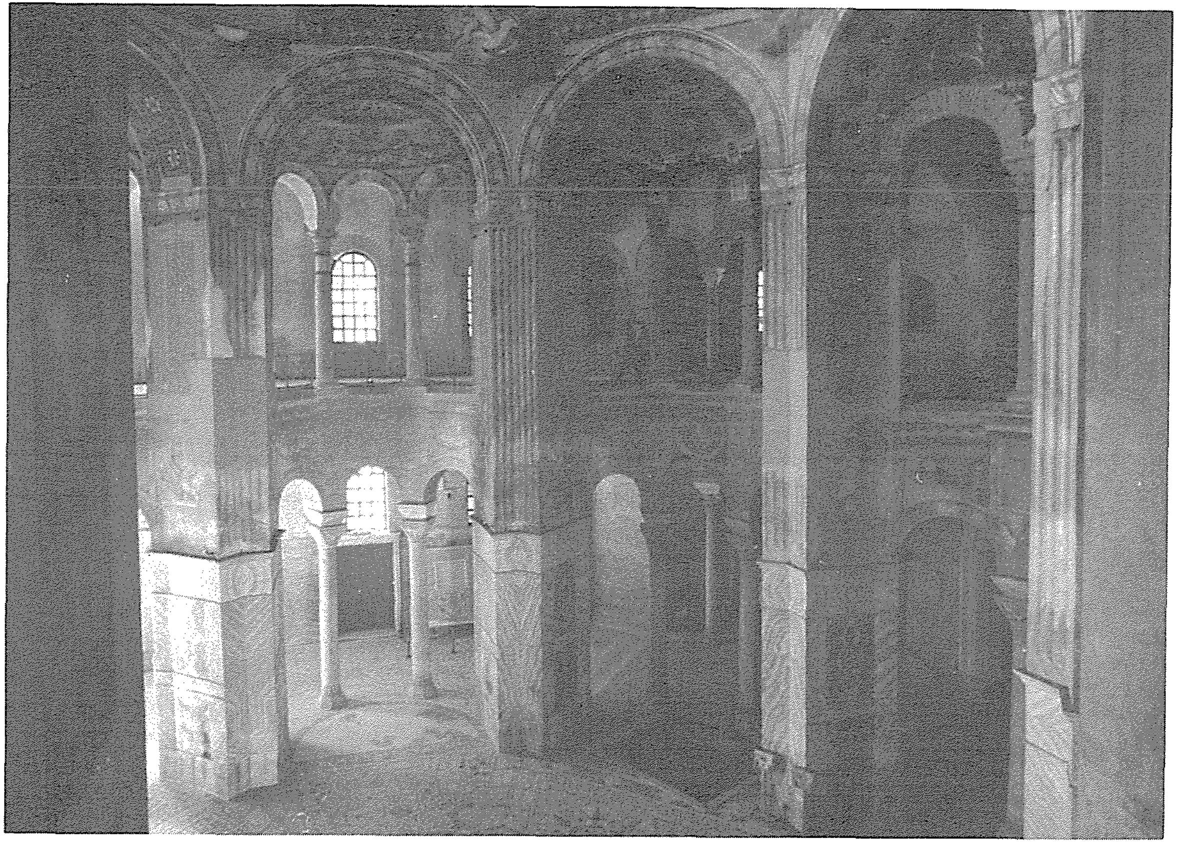

Fig. 11 The beautiful interior of the Byzantine Basilica of San Vitale where a DAG network has been installed

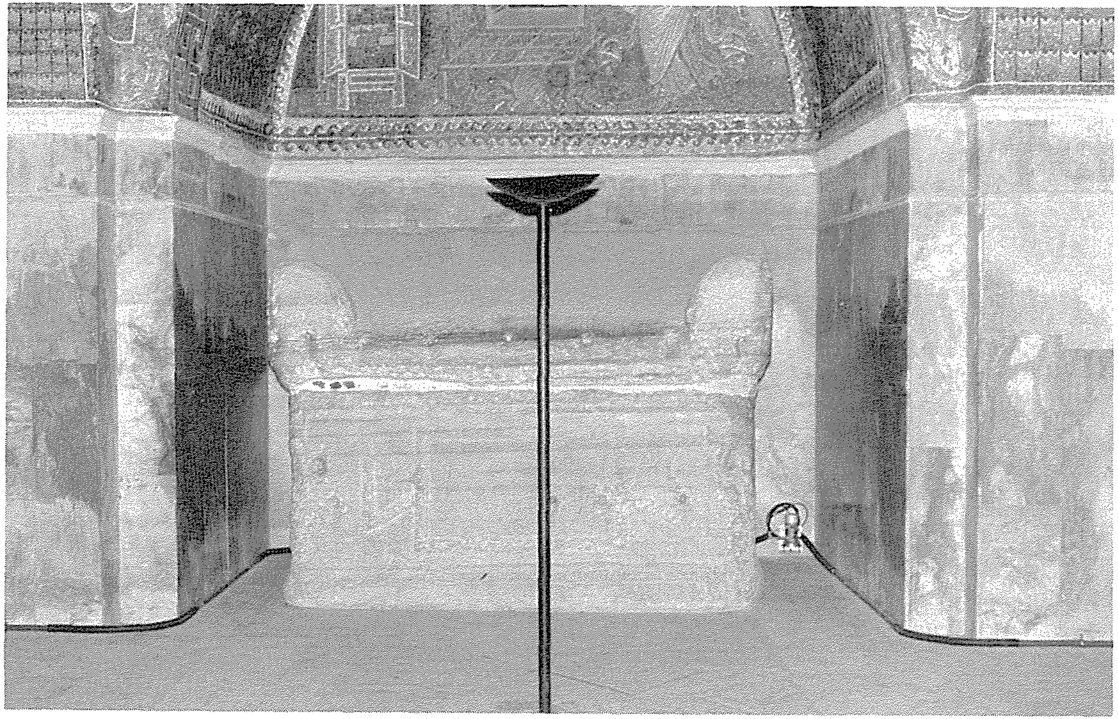

Fig. 12 One of the sensors installed in the Mausoleum of Galla Placidia

and the Mausoleum of Galla Placidia a wide number of instruments, i.e. piezometers and settlement sensors, were installed while inside both buildings (Figs. 11 and 12) a DAG System net work was installed to provide data on the differential settlements of the two structures.

Conclusions

From what above said we can draw the conclusion that a well installed range of accurate and reliable instruments will certainly provide a sufficient amount of data, in a relatively short period of time, to support engineers on choosing the right type of intervention to make. It is obvious that, working on very old structures, great care should be devoted on the type of instruments to install, thus involving sometimes the need of designing new devices with special sensors.

This short report shows the suitability of the DAG System to monitor the stability of ancient structures. Nevertheless, thanks to its great versatility (as explained in above points 1 and 3), the DAG System may be largely used in all branches of civil engineering.

DIFFERENTIATED SUBSIDENCE OF THE UPPER ADRIATIC AREA FACING VENICE (45°11' - 45°21' LAT. N.) AS REVEALED BY 14C DATINGS OF LATE PLEISTOCENE PEATS.

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Summary

Radiocarbon datings of peaty levels from two sedimentary cores provide a first time scale to compare the latest Pleistocene (18,000-21,000 years BP) subsidence of the lagoon of Venice with the continental shelf facing it. From these data it appears that subsidence was very high within the lagoon area and increasingly lower seawards.

Introduction

In the last years a great effort has been devoted by the scientific community to the geological problems connected with the subsidence of Venice. A major question was to understand if and how the subsidence presently affecting the lagoon of Venice was mainly the direct result of human activity through exploitation of the aquifers and settlements or if the area now occupied by the city of Venice was independently subsident for "geological" reasons.

Recent neotectonic work in northeastern Italy (summarized by Zanferrari et al., 1982) reveals that the area under study began to be characterized by a higher subsidence rate with respect to the nearby region, at least from the middle Pleistocene.

The need to quantify the upper Quaternary "geological" subsidence of the Venetian urban area requires the study of Quaternary events at an intermediate time scale. Bonatti (1968) and Fontes & Bortolami (1973) focused their attention to the latest Pleistocene. Through 14C datings of peats and peaty muds interbedded with other continental sediments of long cores drilled at different sites of the plain and lagoon of Venice, they provided useful information about the sedimentation rate of this area. Considering the time span between ca. 23,000 and 18,000 years BP, that is during the last maximum of the Würm glaciation, their evidences show a very high sedimentation rate (3 mm year^{-1}, Bonatti, 1968; at least 5 mm year^{-1} Fontes & Bortolami, 1973).

Aim of the present paper is to provide a new set of datings from the upper Adriatic continental shelf facing Venice, previously unexplored under this respect, in order to estimate the subsidence of the region under study during the last glacial maximum.

During the Ad 78 cruise of the R/V Bannock numerous cores were raised from the subsurface of the upper Adriatic continental shelf (Colantoni

Table 1

Station	Lat. N	Long. E	Wat. depth(m)	Core Length (cm)
Ad 78-150	45°21'.0	12°34'.7	20.0	204
Ad. 78-161	45°11'.3	12°47'.4	29.3	209

& Gallignani, 1980). In the present study we shall take into account two short gravity cores (Ad 78-150 and Ad 78-161) recovered increasingly offshore the lagoon of Venice. Location and characteristics of the two stations are given in Table 1. Basic information about the lithology of these cores is reported by Colantoni et al. (1980) and summarized in fig. 1. Micro- and macropaleontological data of core Ad 78-161 are discussed by Fregni & Borsetti (1980) and Taviani (1980). Both cores reached the continental sediments deposited during the glacial Pleistocene when this part of the continental shelf was a generalized alluvial-lacustrine plain (e.g., Colantoni et al., 1979) as witnessed by sedimentological and paleontological evidences. The continental sequence is truncated by marine sediments representing the Flandrian transgression.

Material and methods

A distinctive feature of the continental sequence sampled by our cores is represented by the presence of peaty levels at different depths.

Although cases of uncertainities, derived from hard water effect (Donner et al., 1971) or contamination with younger illuvial carbon (Black, 1974), have been reported, peat is generally considered as a reliable material for radiocarbon dating (Broecker & Bender, 1972).

Thus, suitable "peats" from our cores were dated with the 14C method in order to construct a chronology of the latest Quaternary events. Results are reported in table 2. Three ages were obtained from peaty levels of core Ad 78-161. The oldest one is 20,110 ± 450 years BP, then we have two younger levels dated back to 19,125 ± 150 and 12,210 ± 165 years BP respectively. Two peaty episodes of core Ad 78-150 provide radiocarbon ages of 20,930 ± 220 and 18,990 ± 270 years respectively.

Discussion

Results obtained from our cores allow a comparison between the geological history of the lagoon of Venice and this sector of the upper Adriatic continental shelf limiting our considerations to the time span from 21,000 to 18,000 years BP.

We consider as Venice standard for our discussion the 30 m core drilled within the lagoon in locality Motte di Volpego (Ascoli, 1966; Cita & Premoli Silva, 1966; Bertolani Marchetti, 1967; Bonatti, 1968) for which two radiometric ages of peat levels are available. The oldest peat is

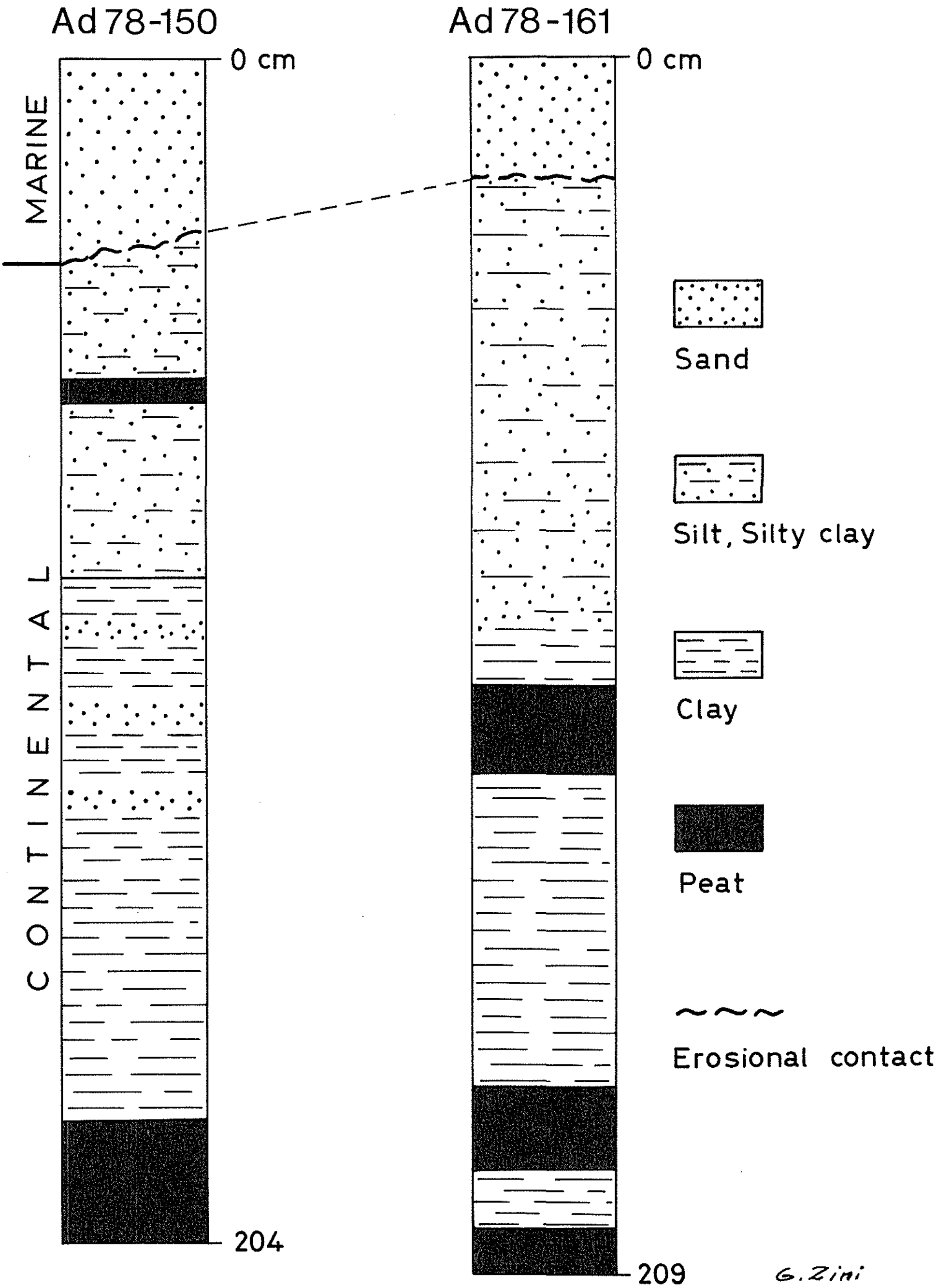

Fig. 1 - Schematic description of cores Ad 78-150 and Ad 78-161.

Table 2

Station and core depth (cm)	Radiocarbon Age (yr BP)
Ad78-150/186-200	20,930 ± 220
Ad78-150/55-58	18,990 ± 270
Ad78-161/202-206	20,110 ± 450
Ad78-161/183-188	19,125 ± 150
Ad78-161/117-120	12,210 ± 165

23,450 ± 500 years BP while the upper one is 19,620 ± 315 (Bonatti, 1968). About 12.4 m of continental sediments, mainly fluviatile sands, are interbedded to these peats and thus a sedimentation rate of ca. 3 mm year $^{-1}$ can be calculated.

The continental sediments (mainly clay) between the peats of core Ad 78-150 have a total thickness of 1.24 m with a consequent sedimentation rate of about 0.6 mm year $^{-1}$. The two lower peats of core Ad 78-161 finally are separated by only 0.1 m of continental clay, the relative sedimentation rate here being ca. 0.1 mm year^{-1}.

Therefore, the comparison of these three estimates would indicate a differentiated sedimentation rate, markedly decreasing seawards (fig.2).

On the other hand, it is evident that our assumption derives from a raw evaluation of such figures and some limits of this kind of analysis must be considered. For example, the time spans of the considered portions of the three cores are not exactly the same. Core Motte di Volpego represents 4,000 years of history vs the ca. 2,000 of core Ad 78-150 and the 1,000 years of core Ad 78-161. The offshore core cover a shorter time span and sediments older than 21,000 years BP were not reached. On the other hand, core Motte di Volpego does not posses peat levels of intermediate age (i.e., around 21,000 years). This fact can be due to unfavourable conditions for the formation of peat although it is also possible that peat formed but was later eroded away by rivers. Thus, the lack of detailed information on the sedimentological history of the study sites introduces a bias in our analysis leaving as unanswered the question regarding the sedimentation rate of the interval between 19,620 and 21,000 years BP in core Motte di Volpego. Additional geological information from the lagoon is needed to clarify this problem. However we want to point out that if at 21,000 years BP peat effectively formed at Motte di Volpego and was then eroded, the sedimentation rate in the lagoon was even higher that previously supposed and, consequently, 3.0 mm year^{-1} is a minimum figure. Whatever the local depositional environment (fluvial, lacustrine or marsh) the meaning of the sedimentation rates as calculated by us is tenable in the light of the absence of tectonic disturbances in between the dated peats.

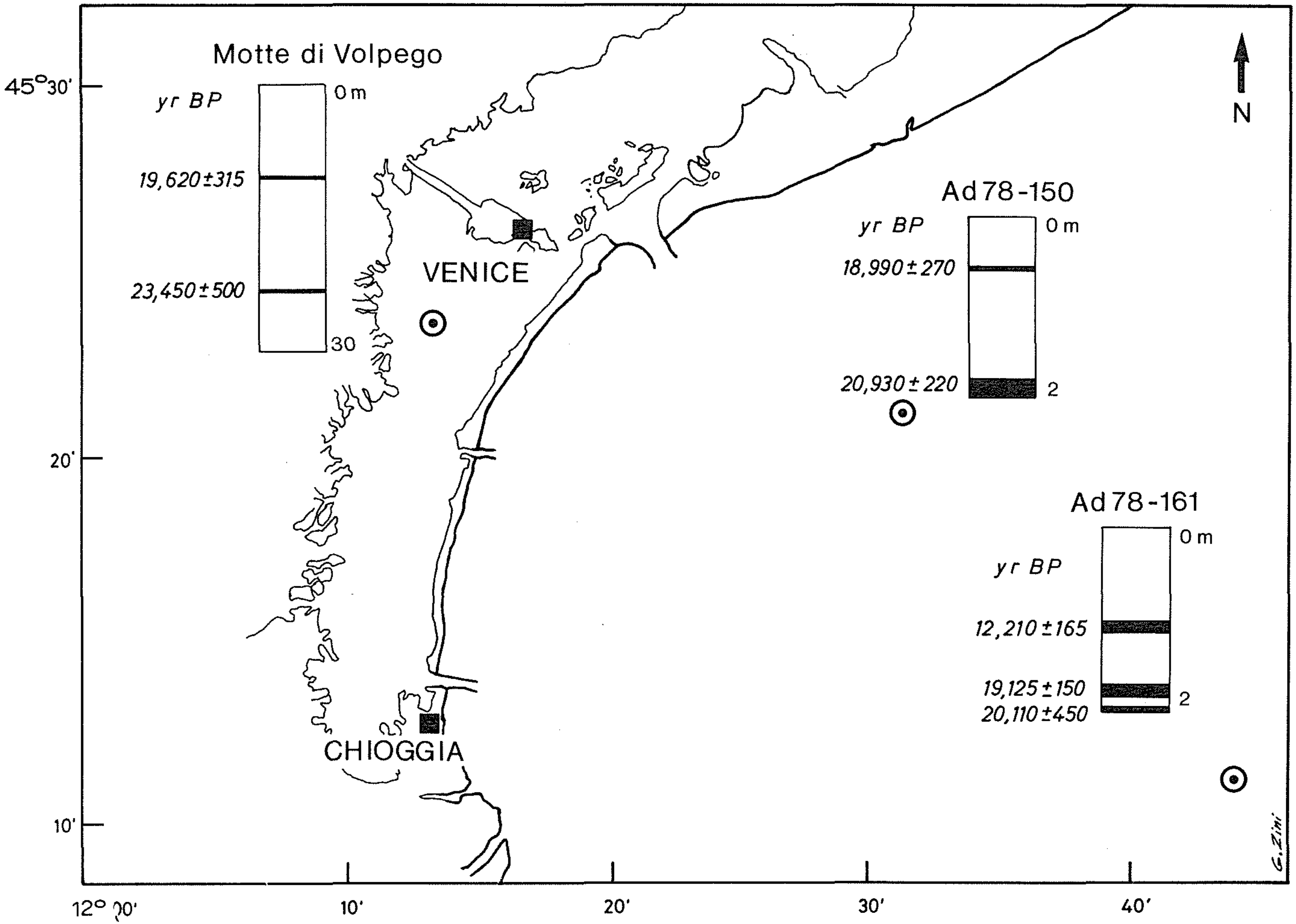

Fig. 2 - Location map showing the position of the cores considered in the present study.

Conclusions

14C ages of peats from sedimentary cores of the lagoon of Venice and Upper Adriatic shelf give evidence of a differentiated sedimentation rate with the highest values in the lagoon and decreasingly lower ones offshore. It can be argued that in the latest Pleistocene (18,000-21,000 years BP) the Venetian area was affected by a very high subsidence with respect to nearby region as the Adriatic continental shelf (45°11'-45°21' N). Our data give a further, indipendent confirmation that already in its preindustrial history the Venetian area was delineated as a very subsident one; on the other hand evidence that the subsidence was not always of the same magnitude has been reported by other authors (Fontes & Bortolami, 1973). Furthermore, it is still open the final origin of such a differentiated subsidence. The sedimentary loading deriving from the discharge in the Venice area of fluviatile sediments from the captured river drainage is probably largely responsible for the observed latest Pleistocene higher subsidence of the lagoon of Venice (feedback mechanism). However, tectonic reasons might be superimposed. The present study is no more than a preliminary approach and additional quantitative data from within and offshore the lagoon are needed before having a complete and coherent picture of the geological problem that is Venice.

Acknowledgements

Thanks are due to Captain, Officers, Crew and Colleagues onboard the R/V Bannock during cruise Ad 78.

Radiocarbon datings were carried out at the University of Miami by J. Stipp. Drawings by G. Zini. Constructive criticism of A. Castellarin, P. Curzi, G.G. Ori, F. Ricci Lucchi and R. Sartori is gratefully acknowledged. Contribution no. 448 of Istituto di Geologia Marina.

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CORRELATIONS BETWEEN SHORELINE VARIATIONS AND SUBSIDENCE IN THE PO RIVER DELTA, ITALY.

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Abstract

This poster shows a sequence of the coastal contours of the Po Delta from the beginning of this century until today, outlined on the basis of cartographical surveys and aerial photographs, and some maps of land subsidence of the last forty years.

The superimposition of the shoreline variations and the land subsidence which occurred in this area stresses the close connection between these two events.

The Po Delta is the extreme manifestation of the complex morphological events which in the arc of about one million years have from the beginning filled the vast area enclosed by the Alps and the Apennines with sediments, thus constructing the Po Valley, about 34,000 sq km and which extends into the Adriatic Sea, giving rise to a projection of the delta.

The formation process of the deltaic system dates back about 2,000 years. At that time, in fact, the beach line extended along the beach ridges of sand dunes still visible on the Chioggia-Ravenna line. The continuous contribution of sediment yield from the river seaward has progressively determined the present projection into the sea of the deltaic system.

Although it is much less than in the past, the sediment yield is more conspicuous today both in coarse (sand) and in fine-grained material (silt and clay). It is believed that the distribution in the sea of this material involves, for the finer suspended material, a strip which extends for about 80 km towards the open sea, with its front at the point of Maistra reaching almost to the Istria coast.

From the pre-Etruscan period up to about 1600, seven beach ridges of sand dunes have been individualized (Ciabatti, 1967) corresponding to shorelines in different times, on which ten cuspate deltas of various arms of the Po lie.

The deltas after 1600 are lobate; thus, the entire Po River Delta zone has been characterized in the past by two phases of deposition (cuspate and lobate deltas) with different growth rates.

Compared the width of the strip, in which the cuspate deltas develop, with the remaining distance to the sea, we immediately have an idea of how different the rate of growth was before and after 1600. In fact, the mean velocity increase from the Etruscan period (about 6th century B.C.) up to 1600 is estimated about 450 m/century. For the last 350 years instead, considering the 25 km which separate the 1600 shoreline from the present one, it is about 7 km/century.

This study shows the evolutional trend of the Po River Delta from the beginning of this century and its subsidence, trying to correlate the two processes and to explain some anomalies of the shoreline observed in the last decades. Particular emphasis is given to the changes of the outermost strip of bars and lagoon from Po di Levante to Sacco di Goro.

The aspect that most clearly distinguishes the Po Delta from the neighboring shores, is the projection into the sea as a result of a continuous and heavy supply of material from the river. Up to the beginning of this century the rich supply of fluvial sediment caused the shoreline to progress constantly. From that time the accretion rate has been slowly decreasing due to the diminished fluvial sediment yield. In spite of this, from 1902 to 1953 the shoreline of the delta advanced, even if with decreasing speed, by creating bars near the river outlets. By connecting with each other, they finally surrounded further coves that were progressively filled until they became land (Fig. 1a and b).

The coastal survey of 1964 (Fig. 1c) shows, in contrast to previous observations, a widespread disappearance of the coastal strip where many ponds again appear, separated from the sea by sandy bars. This situation still remains, even though new bars near the main outlets of the river, already observed in the survey of 1975, clearly appear in the 1983 one (Fig. 1d). It means that sediment yield still takes place, but it cannot counterbalance the shoreline regression and the submerged land; which displayed their seriousness through the 1964 survey.

This upset cannot only be ascribed to the diminished supply of fluvial sediment but it should be considered a consequence of the man-induced subsidence occurred in this area after 1950. Indeed, in the first half of the century natural agents were the main cause of subsidence, that moreover was counterbalanced by nourishment from solid transport and could not cause notable damage. But after 1950, subsidence due to intense extraction of methane-water created concern and a threat to the environment.

The extractions, begun around 1940 with very modest quantities, assumed much greater dimensions after 1950 and involved the progressively deeper aquifers. In particular, the five strate between 100 m and 600 m deep were rashly exploited, causing serious piezometric declines of over 40 m in a few years (Colombo, 1973).

Between 1900 and 1957 land subsidence totalled about 45 cm in the most unfavorable cases at the extremity of the delta with a notable slope towards the east (Caputo et al., 1970). Unfortunately, detailed information does not exist for the first half of this century for the heart of the Delta, nor for the external stretches. There are good reasons to believe that early in the fifties subsidence due to methane-water withdrawal started upsetting the environment. In fact between 1950 and 1957, in the heart of the delta, the heaviest sinking rates in the century were recorded (about 30 cm/y) (Puppo, 1957). The contourlines of equal subsidence for 1958-1967 emphasizes the phenomenon (Fig. 2). As pointed out above, the most obvious drawback of the shoreline appeared at this time and specifically where coastal subsidence was greater (along the Po della Pila and between that area and Po di Tolle).

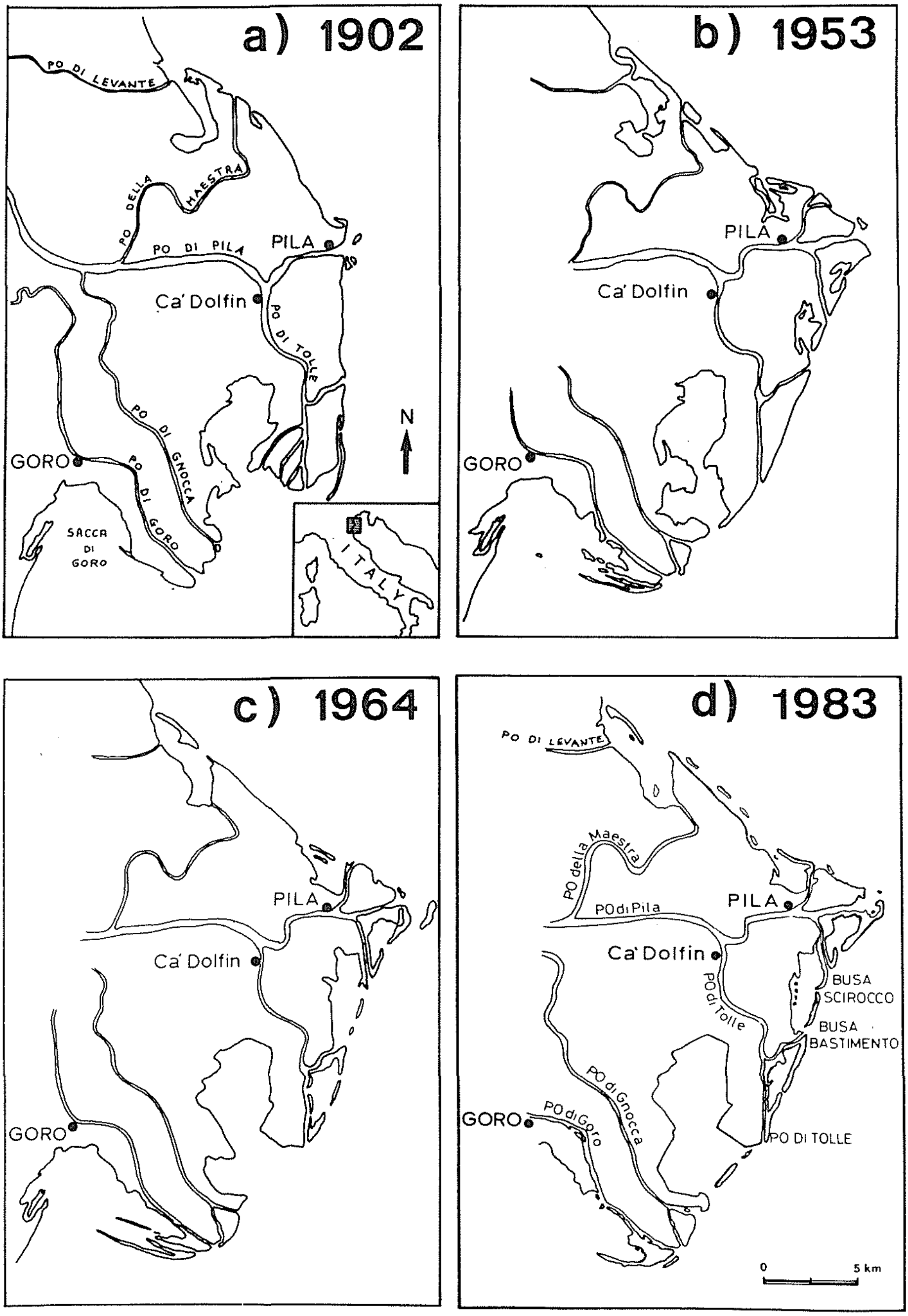

FIG. 1 - Evolution of the Po River Delta during the present century (excerpt and updated after Marabini, 1982).

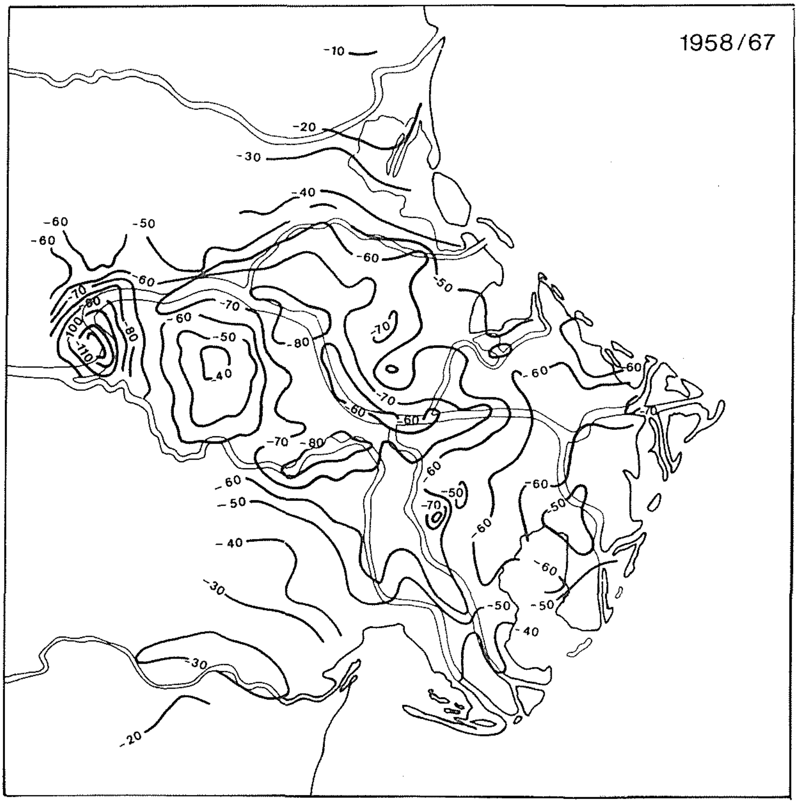

FIG. 2 - Subsidence in cm for the period 1958-1967 (reproduced after Caputo et al., 1970).

In figures 3 and 4 land sinking from 1950 to 1970 is reported along two (dotted) lines spanning the Delta area. Linea C, crossing the inner zone where the removal of fluid from subsoil was heavier, shows greater subsidence (235 cm); line D, which runs closer to the shoreline and along the Po di Tolle, shows here land subsidence from 68 to 153 cm. The latter figures are less impressive, but they rise a greater concern since they refer to a much more delicate area.

Comparing the surveys of 1953 and 1964 (see Fig. 1), wide areas of the beach strip are flooded from Po di Pila to Po di Tolle: in particular the stretch between "Busa Bastimento" and Po di Tolle, which was a center of agricultural activity, is today completely submerged. Further south the disappearance of the bars which separated the Sacca di Scandovari from the sea, brings this coastal zone back to a situation more or less analogous to that at the beginning of this century (1911 survey). Even in this cases, examining southern part end of line C and isolines of subsidence in the 1958-67 period, the cause-and-effect relation is observable.

Generally, there is a very obvious correlation between shoreline regression and permanent flooding of certain areas, and land subsidence.

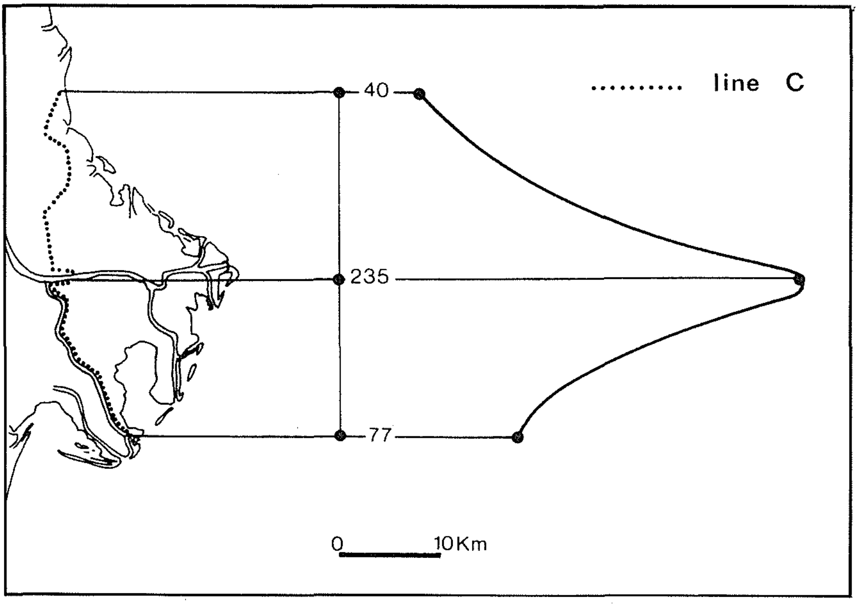

FIG. 3 - Subsidence in cm for the period 1950-1970 along a line (dotted line C after Barbujani, 1973) spanning the central area of the Delta.

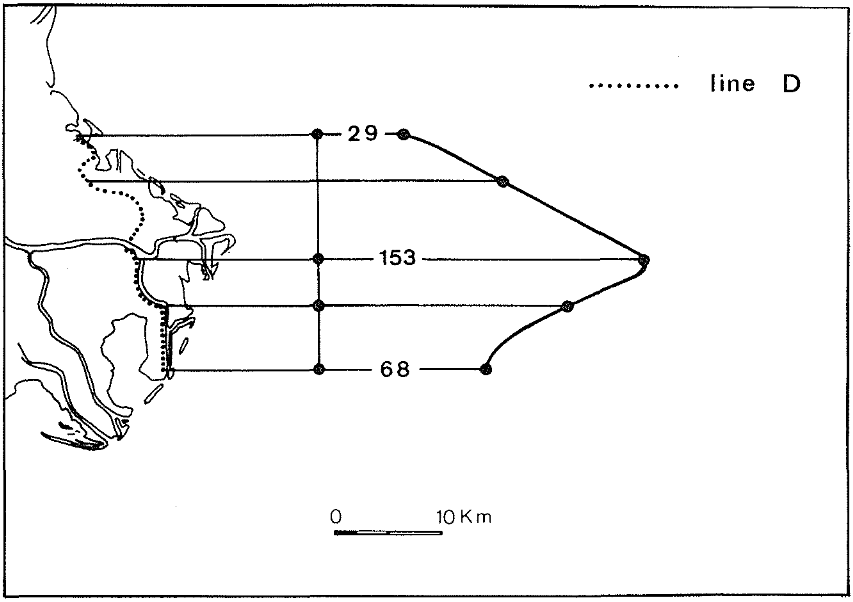

FIG. 4 - Subsidence in cm for the period 1950-1970 along a line (dotted line D after Barbujani, 1973) running close to the Po di Tolle in the southern part.

Because of the nearshore setting and the scarcity of systematic surveys - in any case never carried out for this purpose - it is impossible to give a quantitative correlation between the two phenomena , as it is achievable with a more regular shoreline such as that of the neighbouring Emilia Romagna (Carbognin et al., 1982).

To stress the correlation existing between the two processes there are more recent surveys for both subsidence and coastal topography. They show on one hand the quiescent phase of land sinking (2 cm/y were recorded in 1982) (Bondesan and Simeoni, 1983; Montori, 1983), even though the rates of the first half of the century have not been attained, and on the other hand a new outbuilding at the mouths where fluvial sediment yield is no longer overwhelmed by subsidence.

There remains however a new configuration derived from the upset produced by subsidence which is a mainly irreversible phenomenon.

For the coastline the 1902, 1911, 1929, 1935, 1944, 1953, 1964, 1975, 1983 surveys were used in this study. In particular those of 1902, 1911, 1929, and 1935 were carried out by local agencies and IGM; the 1944 one was obtained by a RAF flight for war purpose; the 1953 survey was made during the GAI flight by IGM; the 1964 one was obtained by a flight for the Delta Padano agency; the 1975 and the 1983 surveys have kindly been supplied by the "Magistrato per il Po - Rovigo".

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LAND SUBSIDENCE OF AN INLAND BASIN, NORTHEASTERN JAPAN — THE YAMAGATA BASIN

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Abstract

The Yamagata Basin is one of the typical inland basins in the northeastern part of Japan. Land subsidence was first noticed around the year of 1967 in the central part of the basin. Geodetic surveys conducted between 1974 and 1982 with reference to triangulation benchmarks showed that the maximum subsidence has reached 28 cm with an area experienced more than 2 cm of subsidence extending over 5600 ha. The land subsidence is considered to have resulted from a large increase in the withdrawal of ground water, decrease of surface water intake areas for subsurface aquifers owing to increasing areal expansion of urban zones of the city, and concentrated withdrawal of deep-seated ground water for irrigation purposes during a short period from May through August.

Introduction

Land subsidence in Japan has first been noticed in the 1920's in areas of large urban centers such as Tokyo and Osaka. The fact that one of the major causes of land subsidence is intensive ground water withdrawal has been documented by the end of 1930's, but such a knowledge has been shared with the general public only recently.

Since the end of World War II, a rapid expansion of economic activities brought about increasing demand for waters for industrial purposes and as a result exploration efforts for and use of ground waters increased steeply. Concomitant with the increasing use of ground water, land subsidence spred from the environs of Tokyo and Osaka to cities in the provinces as well as to other areas where dissolved-in-water type natural gases were abstracted. These areas affected by land subsidence in Japan are mostly located in coastal plain regions (Fig. 1). However, land subsidence in Yamagata Basin, the subject of this paper, is unique in that the problem occurred in an area of inland basin. Because of this unique geographical situation, the Yamagata Basin is of interest and is taken as the subject of discussion in this paper.

From ancient times, the Yamagata Basin has been blessed with abundant supplies of ground water. The urban zone of Yamagata City is situated on alluvial fans which supplied its residents with abundant ground water for daily use. Also, agricultural areas, mostly represented by rice paddies, extend from the distal end of both the Mamegasaki River and River Tachiya fans to a low-lying basin bottom where, in addition to river waters, abundant natural as well as artesian wells supplied waters for agricultural uses. However, increasing demands for city waters created by increasing city population and by a rapid expansion of commerce and industrial activities initiated tapping of abundant ground water resources. This exploitation of ground water caused a westward migration of the zone of artesian wells towards lower levels of alluvial fans and as a result some artesian wells started to dry up. The stoppage of these natural artesian wells necessitated drilling of deep wells for agricultural purposes beginning in about 1952. Droughts occurred in 1955 and 1958 further accelerated this reliance on deep wells. On the other hand, rapid expansions of commerce and indus-

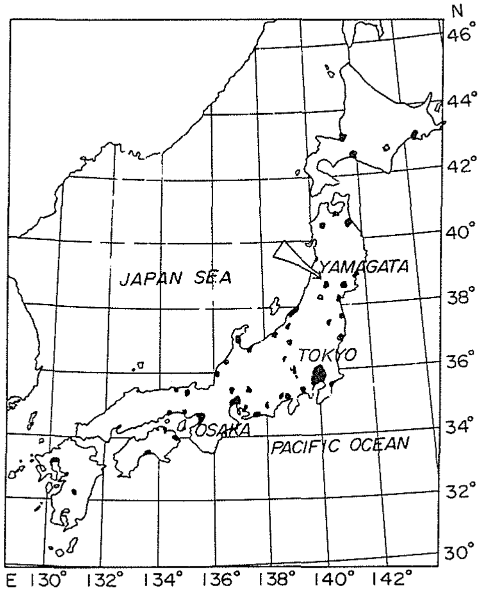

FIG. 1 Areas having land subsidence problems in Japan. Arrow points to Yamagata Basin.

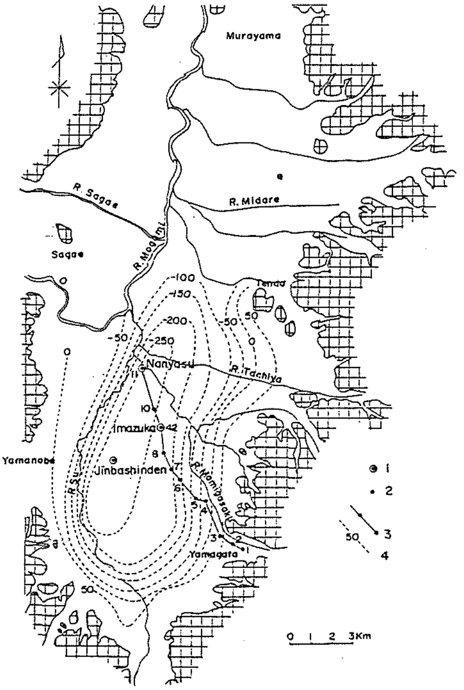

FIG. 2 Depth contours for pre-Quaternary subsurface (elevation in meters relative to mean sea level). 1, Test well. 2, Other active wells. 3, Line of geologic cross-section. 4, Depth contours.

trial activities in this area which started around 1960 concordant with the rapid economical growth throughout Japan, created further demand for ground water. As a result, the number of deep wells increased sharply. A geodetic survey undertaken in 1967 by the Geographical Survey Institute of the Government of Japan with reference to first order triangulation benchmarks along national highway no. 13 showed a maximum subsidence of 5 cm during an 11-year period in the northern part of Yamagata City. At the same time, many deep wells began to show "pop-up phenomena" of well-pipe head relative to the surrounding ground level. A 1973 survey revealed that the maximum amount of well-pipe head pop-up reached 46 cm and the area showing similar subsidence effects spred to a broad region encompassing the northwestern part of the city.

Hydrogeology

The Yamagata Basin is one of the typical inland basins situated in a lowland area between the Ou Backbone Ranges and Dewa Hillyland. The basin having a narrow and elongate boat-bottom shape measures 35 km meridionally and has an east-west width of 15 km. The basic morphology of the basin is closely related to the structural development of this region. Namely, the basin is situated in a synclinorium trending meridionally and the basin configuration reflects structures of geologic formations forming these two mountain chains.

The level of basin floor is 170 m above mean sea level at its southern end and gently slopes down to 80 m at its northern end. This elevation of the basin floor is quite low for such an inland basin in the northeastern part of Japan. A thick body of Quaternary sediments fills the basin and its maximum thickness exceeds 350 m. The pre-Quaternary structure map constructed from subsurface information gained from bore-hole and various seismic reflection data shows a narrow meridional depression in the central part and the southern part has a basin whose elevation stands at 250 m below mean sea level (Fig. 2). The northern part of the basin is filled with a comparatively thin body of Quaternary sediments. This depth difference in the pre-Quaternary subsurface is considered to have derived from differing rates of uplift of pre-Quaternary rock formations. On the eastern side of Yamagata Basin, three extensive alluvial fans are developed along the rivers of Midaregawa, Tachiya and Mamegasakigawa from north to south. On the other hand, there are only a few small-scale fans on its western side. Sedimentary facies of the Quaternary System is reconstructed by using data from deep wells. Fig. 3 shows a cross-section starting from the apex of one of the eastern alluvial fans through the middle and terminal fan segments to the low-lying basin center. Thick layers of sand and gravel containing large boulders are distributed in the upper and middle fan segments. Lithologic sequences of these alluvial fan deposits are intervened by thin muddy layers. Sedimentation of these coarse-grained sediments took place rather discontinuously and these sediments are largely represented by debris flow deposits brought down by numerous floods. Toward the lower part of the fan, these coarse-grained sediments grade laterally to much finer and well-sorted sediments and the basin center is characterized by a mud-dominant facies. The occurrence of land subsidence is centered largely around the area underlain by this mud-dominant facies.

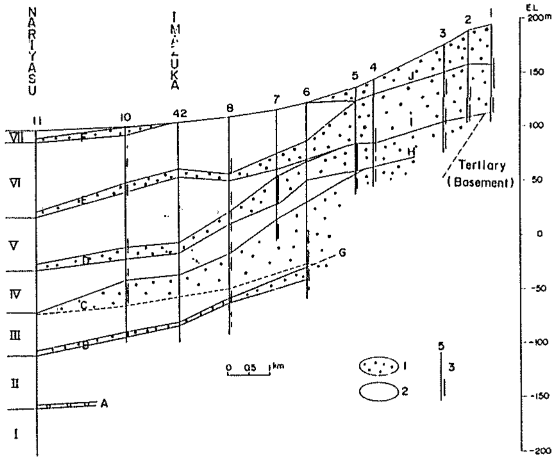

FIG. 3 Geologic cross-section. 1, Sand-gravel-dominant facies. 2, Mud-dominant facies. 3, Position of strainer. 5, Number assigned to each well.

As can be seen in the geological cross-section, aquifers in the alluvial fan areas are sand and gravel layers occasionally containing large boulders. In the basin center, aquifers are sheets of sandy gravel intercalated with the mud facies and the ground water in these aquifers are confined.

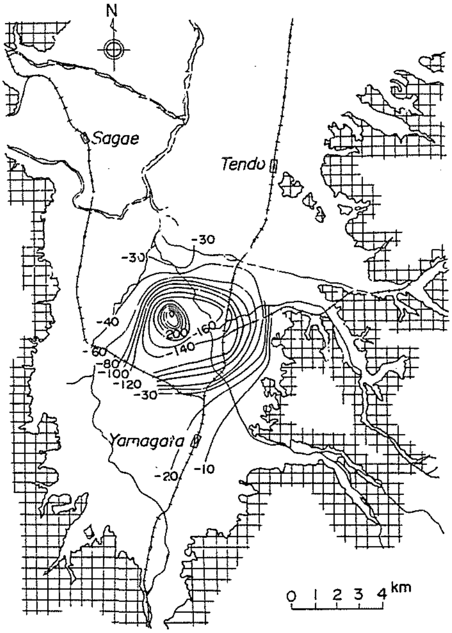

FIG. 4 Subsidence contours between 1974 and 1982. Contour intervals in mm.

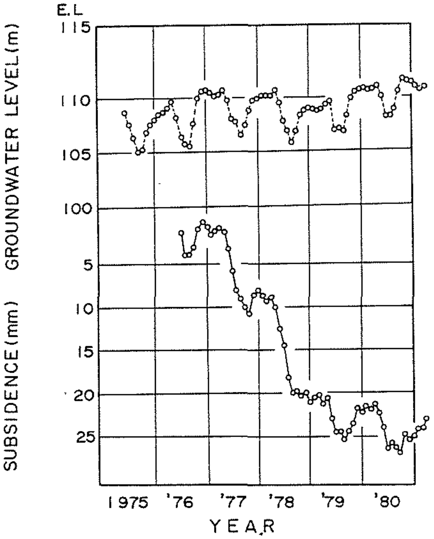

FIG. 5 Comparison of ground water level variations with land subsidence as measured in observation well.

Land subsidence

In the Yamagata Basin, geodetic leveling was carried out once a year since 1974 (Fig. 4). The date of November 1 was chosen for the measurement because this time of the year generally falls within a time period of minimum land movement. Those areas where cumulative amounts of land subsidence between 1974 and 1982 are very large include Nagaomote (No. 39) with its subsidence value of 276.3 mm and Hachimanmae (No. 14) with a value of 224.3 mm. The maximum value of 281.7 mm was observed at Hattori (No. 15). Subsidence contours for a 1974-1982 period exhibit a concentric pattern with the maximum sinking area of Hattori and Nagaomote taking its center. A large area covering as much as 5600 ha experienced subsidence of more than 2 cm. However, the situation is improving since 1978 when restriction of ground water pumpage came to be enforced as a result of the passing of a law aiming at appropriate control and regulation of ground water uses.

The rate of well-pipe head pop-up relative to the adjacent ground level has also been measured once a year since 1974 in November. This survey is carried out by measuring the amount of vertical offset at certain fixed points between the well-pipe head and water-tank or between the well-pipe head and wall sidings of pump-house. This survey shows the maximum offset value from the well's completion date to the year 1982 amounted cumulatively to 50.4 cm at Hattori well. Other large values are 45.5 cm and 46.2 cm. A close correspondence is noted between contours of the vertical offset values and the subsidence rate contours drawn from geodetic leveling, both showing a similar concentric pattern with the highest values clustering at the center. The rate of land level change measured with reference to observation wells is closely correlated with the rate of water withdrawals for irrigation purposes, although a small time lag is noted. A rapid land sinking occurs from the middle of May to nearly the end of September and the land rebounds a little thereafter although not recovering to the previous level. The net subsidence shown is therefore the annual subsidence rate minus this rebound rate. Fig. 5 shows the magnitude of annual rate of subsidence measured at No. 2 observation well. The maximum subsidence of 13.84 mm occurred in 1978 and the minimum of 5.27 mm in 1980. The largest net subsidence amounted to 12.99 mm in 1978 and the smallest was 1.23 mm in 1979.

Uses of ground water

Concomitant with the stoppage of flows of natural as well as artesian wells which became noticeable around the year 1951, ground water withdrawals were started by drilling deep wells around the terminus of the alluvial fans. The number of deep wells increased every year and at present more than 60 wells are active. Fig. 6 shows annual changes in the rate of ground water uses from 1952 to 1982. The total amount of ground waters used in the year

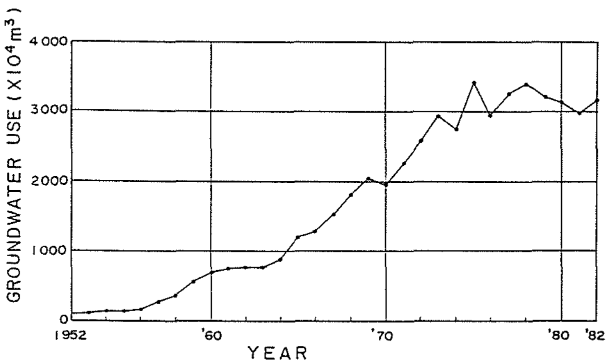

FIG. 6 Rate of ground water uses in Yamagata City from 1952 to 1982.

1982 amounted to 31.32 million cubic meters. Of these, industrial consumption totaled 14.69 million cubic meters (42.8%) and agricultural uses 11.75 million cubic meters (32.6%). Demand from commerce and service industries amounted to only about 10%. Monthly rates of consumption can be seen in Fig. 7 and show a very unbalanced seasonal distribution with 95% of agricultural uses occurring in a 4-month period from May to August. On the other hand, consumption by other categories of activity is nearly constant throughout a year.

Annual precipitation, rates of ground water withdrawal, and land subsidence

Variations in annual precipitation influence greatly the rate of ground water withdrawals. The annual precipitation for the year 1975 was 805 mm and rainfall during the irrigation period from May to August of the same year was only 235 mm. The total amount of ground waters pumped for this year reached 34.37 million cubic meters, of which 17.91 million cubic meters were for agricultural uses. This volume was the largest ever withdrawn for irrigation purposes. In contrast, the year 1980 had an annual precipitation of 1439 mm, of which 724.5 mm fell between May and August. The total ground water pumpage for this year was 31.32 million cubic meters and waters used for agricultural purposes were 11.75 million cubic meters. This amount was 65% of that used in 1975 (Fig. 8).

These observations indicate that years having a large precipitation show

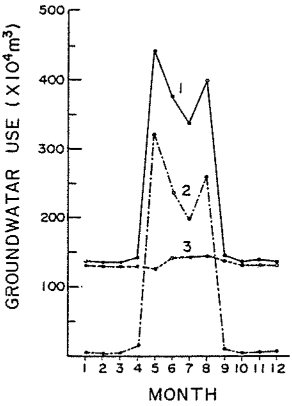

FIG. 7 Monthly rate of ground water uses for the year 1974 in Yamagata City.
1, total amount of uses.
2, agricultural uses.
3, other categories uses.

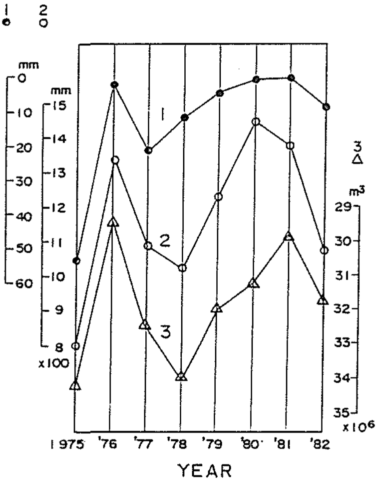

FIG. 8 Interrelationship between rate of annual average land subsidence (1: filled-in circle), annual precipitation (2: open circle), and rate of ground water withdrawal (3: triangle).

a small drawdown owing to the lesser amounts of water withdrawn for irrigation. A similar interrelationship has been observed with the rate of land subsidence. While the annual rate of land movement was quite large in 1975 amounting to 53.0 mm, it was only 0.8 mm for 1980. A similar correlation also exists with the rate of vertical displacement of well-pipe head relative to the surrounding ground level.

Conclusions

The main responsibility for the land settlement in the Yamagata Basin can be ascribed to the following:

(1) The urban zone of Yamagata City is situated on alluvial fans. With increasing rate of expansion of the urban zone, surface water intake ares for the subsurface aquifers steadily decreased.

(2) Agricultural areas (rice paddies) are mainly distributed over the terminus area of alluvial fan and the juxtaposed, low-lying flat plain. Because of the reason given above, decreasing amounts of water flowing out of natural and artesian wells placed an increasing reliance on deep wells for deep-seated ground water.

(3) The agricultural areas are underlain by muddy incompetent sediments intercalating occasionally with thin sheets of sand and sandy gravels.

(4) Because of a combined effect of decrease of surface water supplied to the subsurface aquifers, a seasonal concentration of intensive water withdrawal, and occurrence of droughts, land subsidence started as a result of dehydration and compaction of the underlying muddy strata.

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GROUNDWATER RESOURCES IN JAPAN WITH SPECIAL REFERENCE TO ITS USE AND CONSERVATION

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Abstract

In Japan, the Industrial Water Law in 1956 and Building Water Law in 1962 were established to change the use of groundwater to the use of supplied surface water in order to check severe land subsidence. Also, many local governments promulgated ordinances to restrict the use of groundwater to stop subsidence in 1970's. These laws and ordinances contributed to the calming down of the subsiding phenomena in many areas of Japan.

In addition, the groundwater conditions studied for environmental reasons in groundwater basins are useful in conservation and proper use of groundwater without resultant land subsidence in Japan.

General

1. Background and Purposes of the Research

Groundwater is a valuable water resource and an important constituent of the earth. As a water resource it has been valued especially for its good quality, its stable temperature and its economical availability. Groundwater has played an important role in the industrialization and the socio-economic development of Japan.

Recently, the quantity of the groundwater pumped in the whole country amounted to about 14 billion cubic meters a year. That is equivalent to about one-sixth of the whole fresh water supply (Table 1). Because there exists a limit to the amount of natural recharge of groundwater, the excessive withdrawal of groundwater beyond the limit has brought about many serious problems. Among the problems are increasing difficulty in pumping groundwater, salinization of aquifers along coastal areas, and land subsidence due to lowering groundwater levels. Land subsidence especially has caused damage not only to buildings directly, but also has raised the degree of danger from flooding and inundating over the areas of low-lying lands at the time of high water and high tide. Land subsidence also has remarkably lessened the utility value of the national land.

In order to improve this situation, the Resources Council publicized an October 1974 report of investigation concerning the conservation of groundwater and its use. In this report, the Council recommended such basic administrative measures as establishment of a synthetic management system, promotion of scientific and technological research, and synthetic arrangement of the laws for groundwater management. Though some improvements of the situation have been made since then, many problems remain unsolved. Therefore, the Council conducted further detailed investigations into the actual state of the conservation and effective use of groundwater in order to make additional definite recommendations.

The Council set up a subcommittee headed by Dr. Soki Yamamoto, which consists of 20 members of officials from concerned government organizations

Table 1.--Use of Groundwater in Japan

(unit : 100 million m^3/year)

Purpose / Classification	Industrial	Domestic	Building water supply	Agricultural	Total
Water supply (A) (%)	182.8 (20.9)	123.4 (14.1)	7.9 (0.9)	570.0 (65.0)	876.2 (100.0)
Ground water supply (B) (%)	72.5 (52.0)	29.5 (21.1)	7.9 (5.7)	37.5 (26.9)	139.5 (100.0)
Dependence on groundwater (%) (A/B)	39.7	23.9	100.0	6.6	15.9

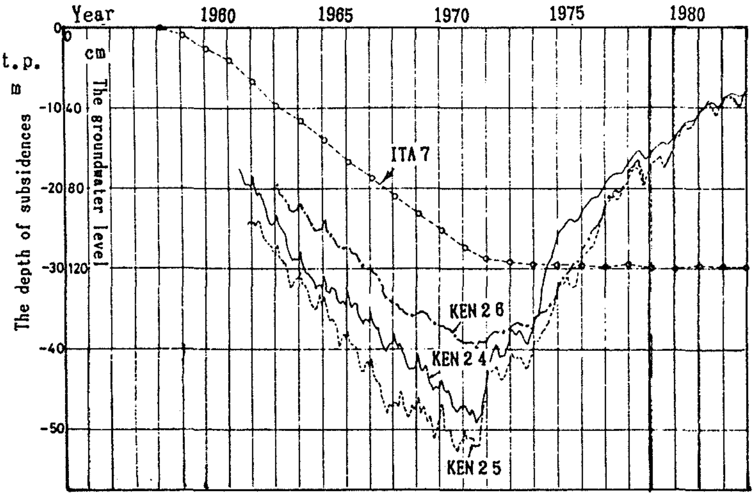
1. The data on industrial water and domestic water are based on "The program of the long-term water supply", (1978), by National Land Agency.
2. The data on the water for agriculture is based on "The actual investigation on the use of groundwater for agriculture", (1978), by The Ministry of Agriculture, Forestry and Fisheries.
3. The data on the building water supply is based on "A general condition of land subsidence areas occurring in the whole country", (1978), by Environment Agency.

and specialists from universities and research institutes. The committee analyzed many data and information on groundwater use and conservation, which was collected from various national and local governments, and reached some important conclusions from the research.

2. Results of the Research

(1) The actual state of land subsidence: In the areas where serious land subsidence continues, the restrictions of groundwater use based on Industrial Water Law (1956) and Law concerning Regulation of Pumping-up of Groundwater for Use in Buildings, the so-called Building Water Law, (1962) have been applied. But to realize further improvement of the situation, many local governments have made additional efforts, such as enactment of ordinances to strengthen the regulation and coordination of groundwater uses in their territories and the development of alternative water resources (Fig. 1, Fig. 2). Owing to these efforts, the annual rate of land subsidence is not as large at present as it was. However, the number of land subsiding areas has been increasing. The Environment Agency reported that land subsiding areas increased in number from 46 in 31 prefectures in 1974 to 60 in 36 prefectures in 1982. In some areas, the annual rate of land subsidence amounted to 5-10 centimeters. Moreover, the number of areas having salinization of groundwater due to sea water intrusion has been increasing remarkably from 14 areas in 1974 (Fig. 3, Fig. 4).

(2) Promotion of the research concerning new methods for the management



Bench-mark ITA 7 *Itabashi ward Shingashi 2 chome*
Well: KEN 2 4 TODABASHI No1(258-268m)
Well: KEN 2 5 TODABASHI No2(103-113m)
Well: KEN 2 6 TODABASHI No3(51- 59m)

Number in ( ) signifies the depth of the strainer from the ground level

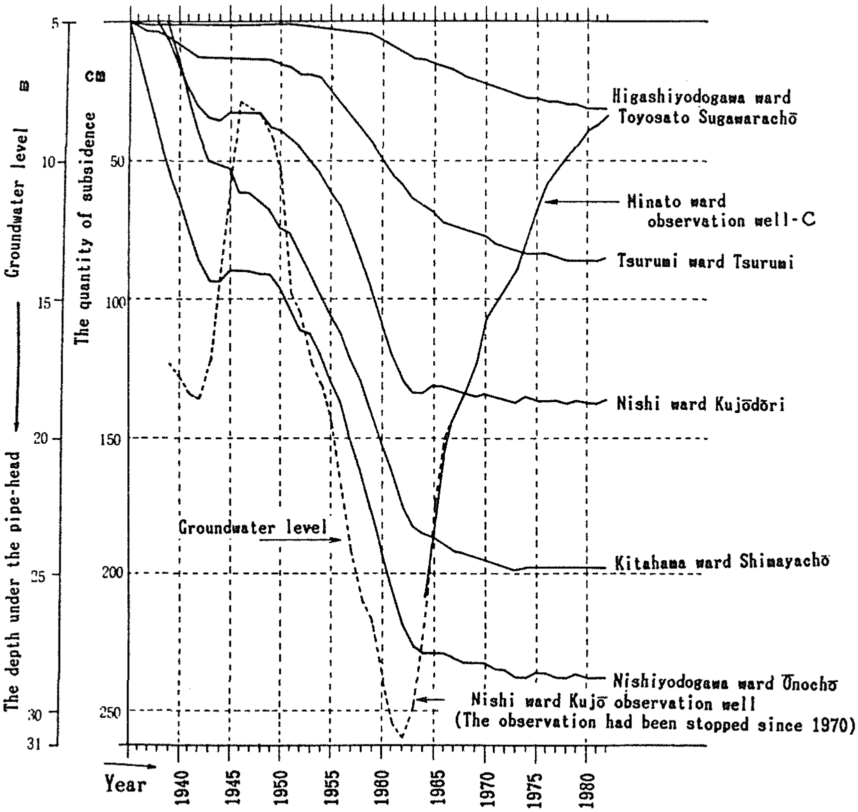
Fig. 1--Changes in Groundwater Level and Total Depth of Land Subsidence at Itabashi Ward (Tokyo, 1982).

of groundwater: In order to prevent land subsidence, regulations respecting the depth and the sectional area of well (withdrawal pipe) have been imposed in the designated land subsiding areas since 1956. However, for ensuring better management of groundwater, there is an urgent need for adding data on groundwater level as a new index of synthetic control over groundwater uses in the specific groundwater basin as a whole. For this reason it's necessary for the various departments of government and each local public body to promote a systematic water-level observation program.

(3) Development of new technology concerning the conservation and the utilization of groundwater: In order to conserve and utilize groundwater positively it is necessary to promote research and development including the experimental works for the advancement of the technology of groundwater recharge and development of groundwater reservoirs (Fig. 5, Fig. 6).

(4) Utilization of groundwater as water resources for emergency use: Groundwater has to be recognize as fire-extinguishing water or as drinking water in an emergency or severe calamity, such as an earthquake. Groundwater is especially important as drinking water. Therefore, to ensure adequate water at the time of a disaster, groundwater supply systems for emergency use must be constructed and maintained in large cities (Table 2, Table 3).

(5) Thermal utilization of groundwater: Recently highly efficient heat-pumps have been developed, so it may be possible that groundwater will be utilized as the heat-source for air-conditioning apparatus and the thermal utilization of aquifer will increase. From the view point of



The end of the Second World War, August, 1945.
The beginning of water supply by the first industrial water supply facility. June, 1954.
The enforcement of Industrial Water Law. June, 1956.
The enforcement of the land subsidence prevention ordinance in Osaka. April, 1959.
The beginning of water supply by the second industrial water works. May, 1959.
The beginning of water supply by the third industrial water works. September, 1961.
The enforcement of Building Water Law, and Industrial Water Law (revised). August, 1962.
The beginning of water supply by the fourth industrial water works. October, 1964.
The beginning of water supply by the fifth industrial water works. October, 1965.
The end of the period to permit the withdrawal of groundwater for the industry at the designated area in the city. December, 1968.

Fig. 2--Changes of Groundwater Level and Total Amount of Land Subsidence in Osaka City
(Osaka conference on the measures for land subsidence control, 1981)

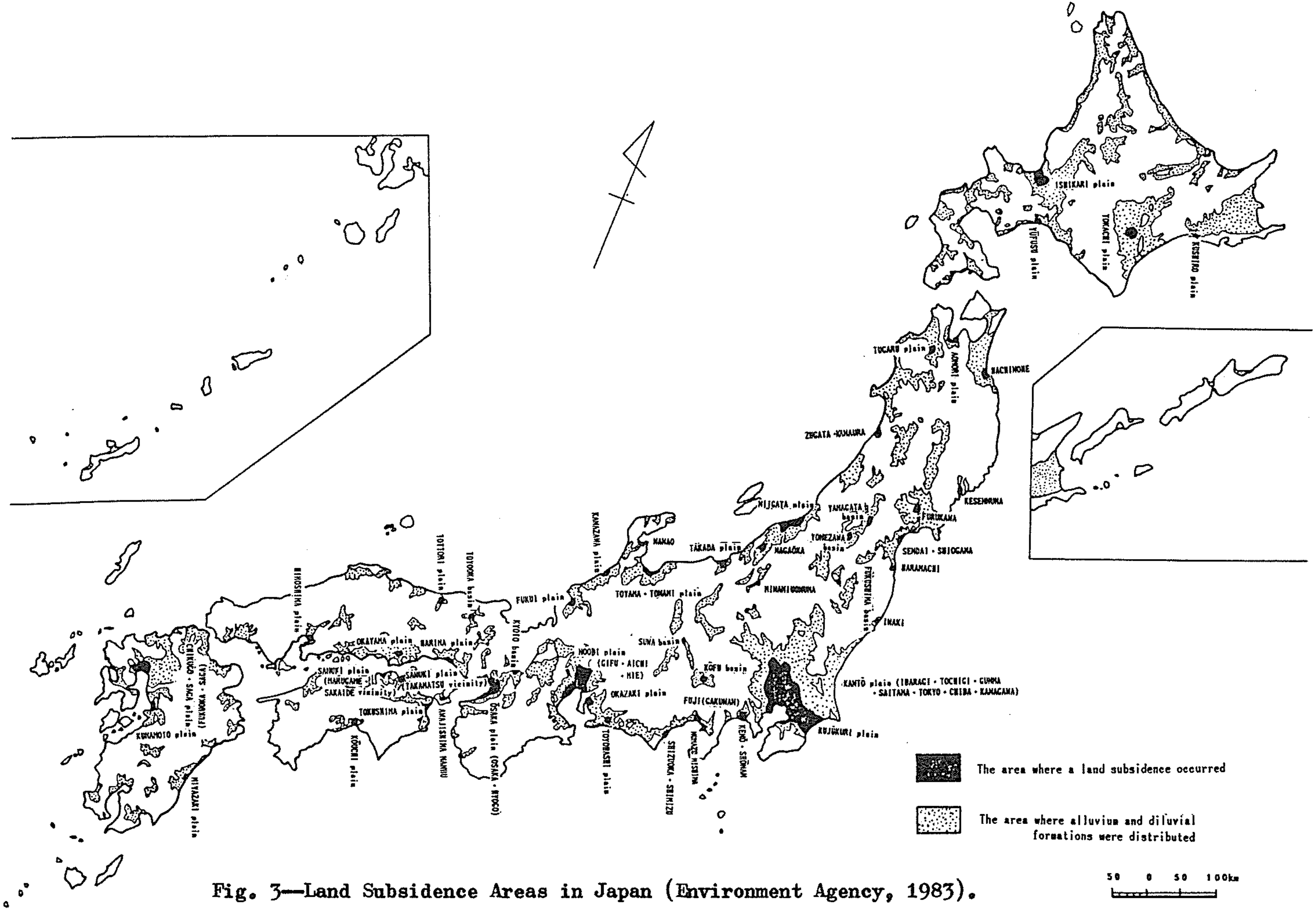


Fig. 3—Land Subsidence Areas in Japan (Environment Agency, 1983).

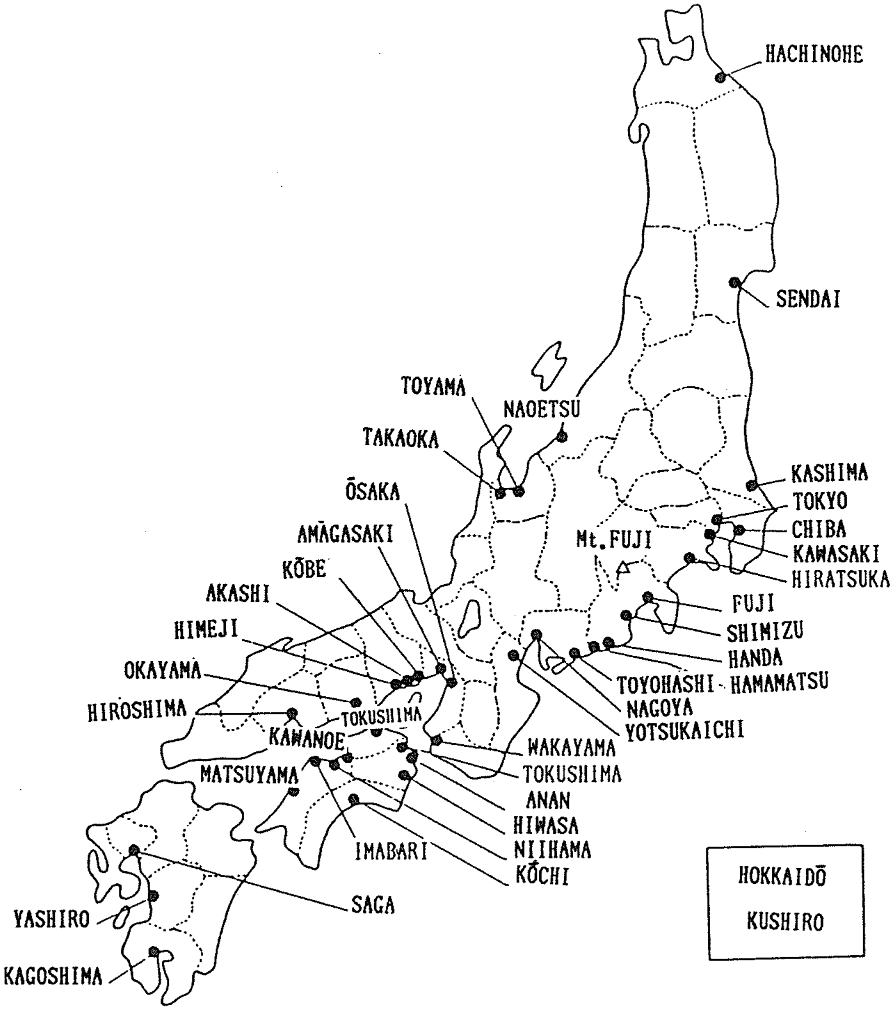


Fig. 4—Areas where Sea Water Intrusion Occurred
(exclude HOKKAIDO) (IKEDA, 1982)

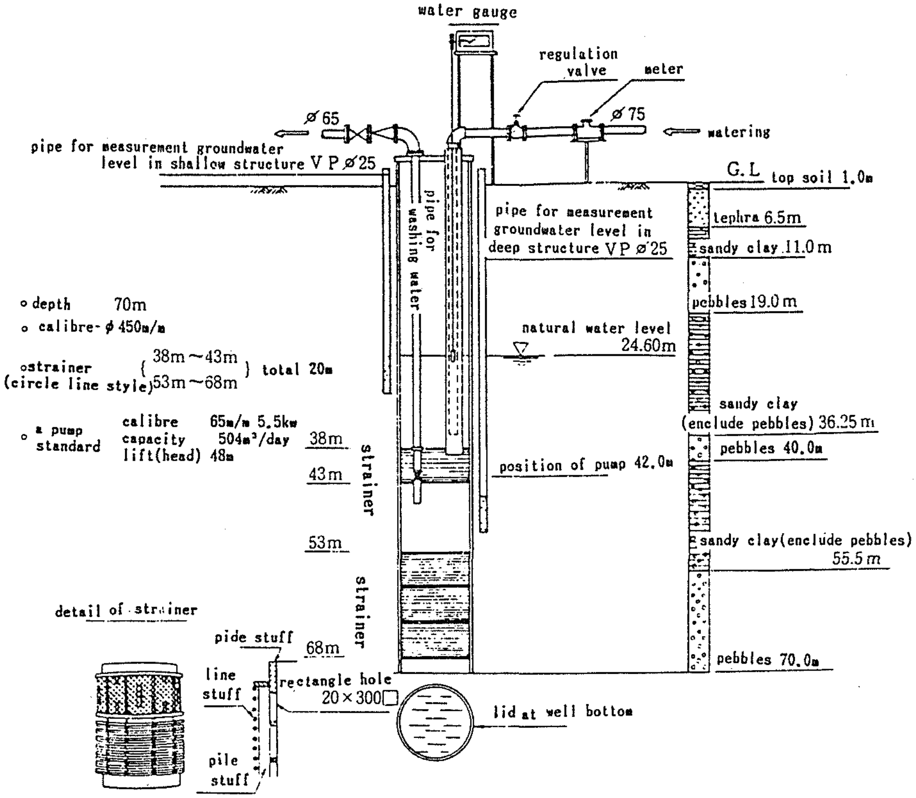


Fig. 5--An Example of Artificial Recharge of Groundwater (Hadano City) (From the data of Hadano city)

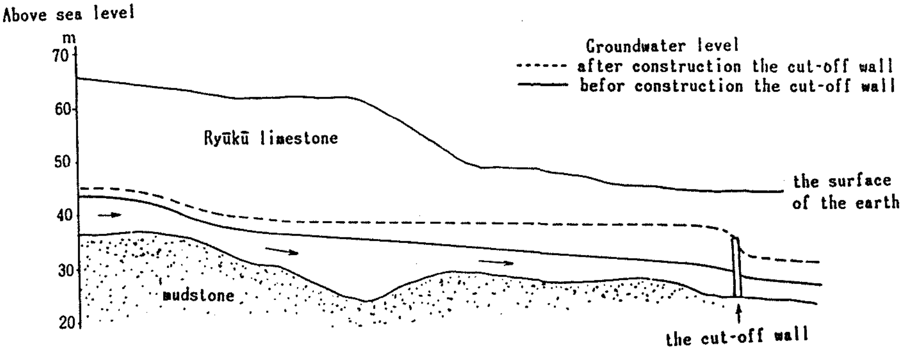


Fig. 6--An Ideal Development of Underground Reservoir

(Cross section of the Groundwater level changes on Minafuku Groundwater Reservoir from the data of Ministry of Agriculture, Forestry and Fisheries, 1981)

Table 2--Groundwater for Emergency Use

Objects of use

- For disaster prevention (particularly for extinguishing the fire)
  It requires much water in a short time. Although the total quantity is rather small, it is necessary to consider the distribution of withdrawal points.
- For public welfare (particularly for the maintenance of life of the people)
  The quantity of the water pumped up per unit time is rather small. There is a possibility that the term of supply is long, for instance it takes several days. The total quantity is rather large.

Classification	Number	Water serving both for fire extinguishing and drinking during an earthquake
Hydrant	798,325	
A water tank for fire protection	266,920	2,678
Well	21,343	127
Total	1,086,588	2,805

Table 3--The Number of Water Utilizations Needed to Extinguish the Fire in the Whole Country and its Items
(from data by the Fire Defence Agency)

effective utilization of natural energy, it is necessary to promote study on the techniques for utilizing thermal energy of groundwater by application of heat-pump systems, as well as to make a guide for the thermal utilization of groundwater so as to prevent land subsidence in advance (Fig. 7).

(6) Strengthening of management systems for groundwater: Groundwater is always a valuable water resource, whether a calamity occurs or not, but if the wrong way is taken to control the groundwater serious difficulty will result. After higher development of land use, the utilization of groundwater will be diversified and the control of both the quantity and quality of groundwater will increase in importance. Therefore it's necessary for local public bodies to strengthen the systems and special organizations for groundwater management.

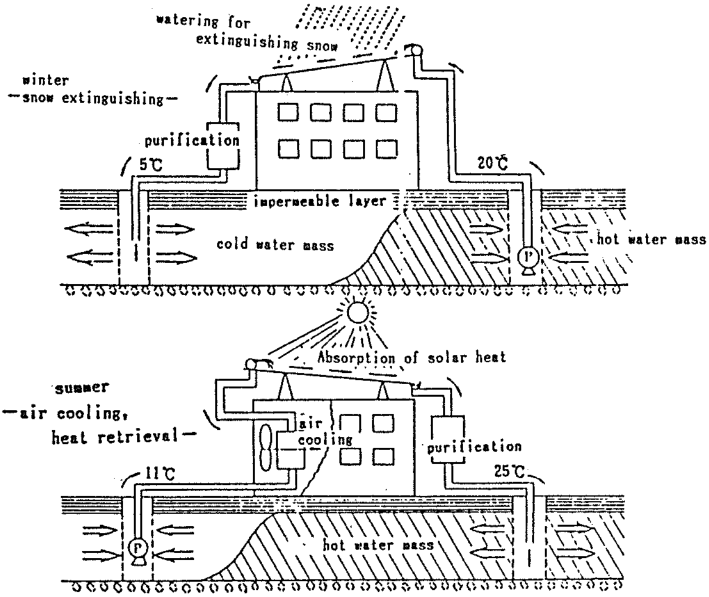


Fig. 7--Thermal Utilization of Aquifer
(Yokoyama, 1979)

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FIELD MONITORING OF SUBSIDENCE EFFECTS IN AIT CAMPUS, BANGKOK, THAILAND

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Abstract

Field monitoring and in-situ tests were carried out to evaluate the amount of subsidence and subsidence effects in the Asian Institute of Technology (AIT) campus. Precision surface levelling were done twice on marking points permanently attached to the structures, surface settlement plates, bench marks, and pavements to observe the absolute subsidence. Inclinometers were installed to observe the horizontal ground movements. The existing compression indicators were monitored to observe the deep and shallow layer compression. In-situ tests consisted of Field Vane Shear tests and Pressuremeter tests were done to find out the field soil properties in the soft clay layer down to 10 m depth. The results show that in three months period, the average settlement of the ground surface is 2.00 cm while the maximum settlement of the building walls range from 2.5 cm to 3.94 cm. The layer compression indicator showed that most of the compression occurred in the upper 10 m layer. The in-situ test results revealed the presence of a thick weaker layer of complicated soft clay layer of higher compressibility in the subsoil below the Central AIT Campus and NZ Housing Area where most of the damage due to subsidence occur.

Introduction

The present AIT campus is about 10 years old. Among AIT buildings and structures, there are widespread indications of differential movements (Bergado, 1983b) evidenced by horizontal and vertical cracks of existing buildings, longitudinal cracks on the asphaltic pavements of roadways, cracks on concrete pavements of tennis courts and walkways, lateral movements towards canal excavations, etc. In this paper, field monitoring not only includes the observations of subsidence, horizontal movements, and subsidence effects but also the in-situ testing of the subsoil deposit for possible correlation of subsoil strength with the surface manifestations of subsidence. Most of the results presented in this paper were derived from the work of Apaipong (1984) under the guidance of the other authors.

Field Monitoring Devices

A hundred locations were selected for fixing the steel rulers as measurement points for observation of the settlement of buildings and walkways. The steel rulers are permanently fixed on the walls and columns of the structures. Five new locations around the campus were selected for installation of surface settlement plates for observation of ground subsidence. Their locations are indicated in Fig. 1. Two locations in the NZ Housing Area were selected for installation of inclinometers (Fig. 1) for observations of horizontal ground movements.

There are three types of existing compression indicators that were installed since 1979 as part of Bangkok subsidence studies (AIT, 1978, 1980, 1981) as station 25 located near AIT campus entrance. These are auger tip, cone tip and deep compression instruments. Both soil compression and pore pressure can be measured by these instruments.

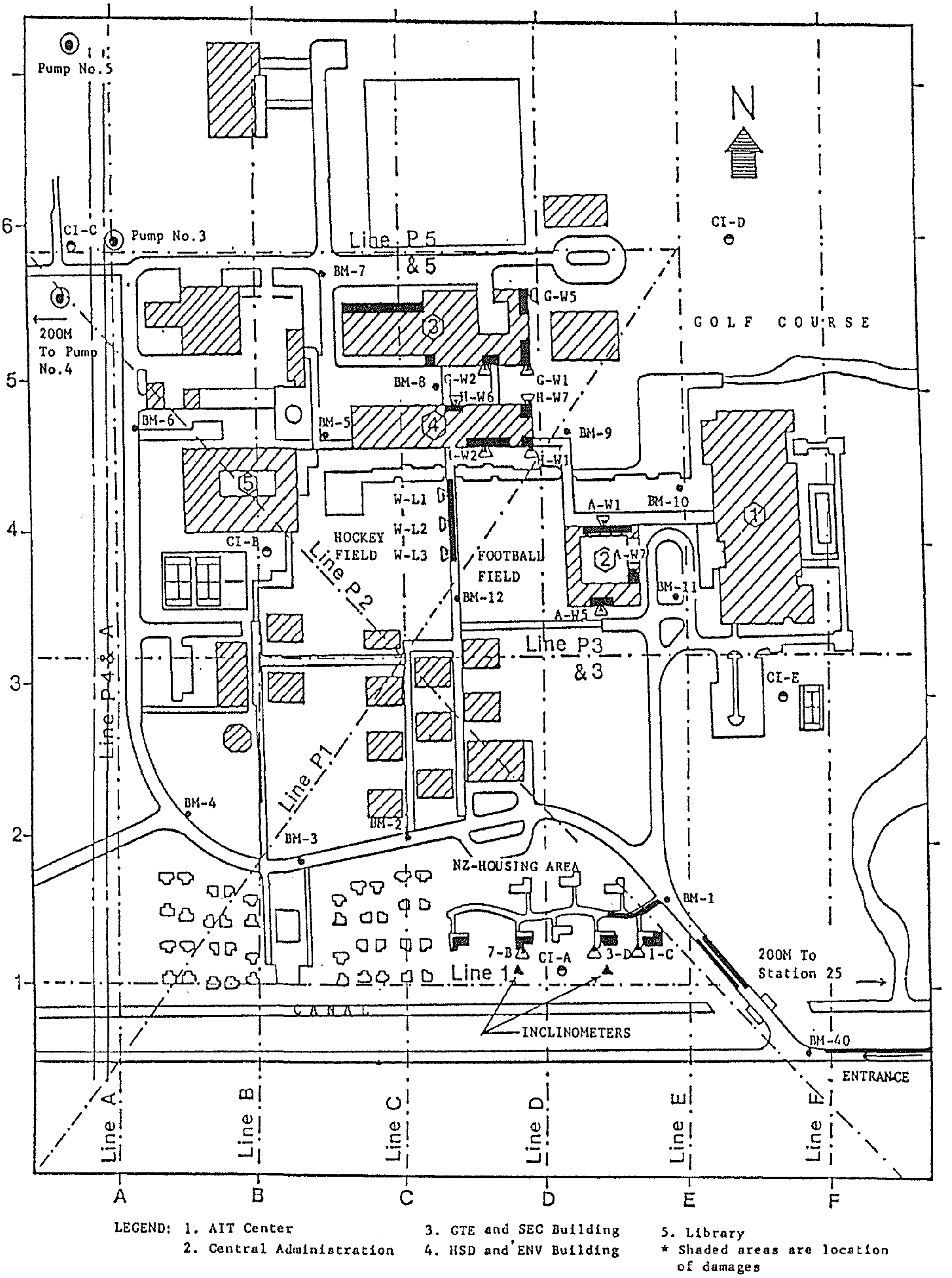

Fig. 1 The Map of Study Area, AIT Campus (Scale 1:4,000)

In-Situ Tests

A Geonor vane test apparatus was used to determine the in-situ undrained vane strength. In this test, a measured increasing torque is applied to the shaft connected to the vane until the soil fails by circumscribing a cylindrical surface. Bjerrum (1972) found the disparity between the actual undrained shear strength from the field and the value obtained from the field vane test such that a correction factor should be applied.

The pressuremeter used in this study is the monocell type manufactured by Oyo Corporation. This type employed a 70 cm long probe and consisted of three main components, namely: the probe, the control unit, and the tube. Three main soil parameters were obtained, namely: the horizontal total pressure at rest (Po), the yield pressure (Py), and the limit pressure (Pℓ). In addition, the pressuremeter modulus (Em) can be obtained in the straight line portion of the pressuremeter curve between Po and Py where the soil is in the elastic state. Gibson and Anderson (1961) suggested the equation for Em which indicates the relative compressibility of the cohesive subsoil.

Land Subsidence in AIT Campus

The land subsidence in AIT campus is mainly caused by groundwater pumping for its water supply. The recording of groundwater pumping began in 1980. During the past four years, the rate of groundwater extraction is given in Fig. 2. The depth from which the quantities of groundwater are extracted is about 200 m in the Nonthaburi Aquifer. The use of groundwater supply appears to have increased rapidly in 1982. It is estimated that at present, the pumping rate is about 4000 m^3/day. The locations of five deep wells are shown in Fig. 1.

The AIT campus is situated within the Lower Central Plain of Thailand on a flat deltaic-marine deposit in an area about 40 km north of Bangkok (Fig. 4). The ground elevation is generally about 1.5 m above the mean sea level (MSL) and the groundwater table fluctuates around 1.0 below the ground surface. The subsoil profile (Chalermsak, 1977; Dum, 1977) as shown in Fig. 3 consists of sedimentary deposits forming alternate layers of clay and sand with gravel down to 1000 m depth. The three uppermost layers consist of 2 m thick weathered clay, about 8 m thick soft clay, and about 5 m thick first stiff clay layer.

The pattern of subsidence occuring in AIT campus due to groundwater pumping is of the form of localized ground depressions in open field areas, differential movements in asphaltic and concrete pavements, and differential settlements between structures on shallow foundations and structures resting on deep foundations. The drawdown of piezometric level in the underground results in an increase in the effective stresses in the soil strata. While the total stress is constant, the increase in effective stress corresponding to the decrease in pore water pressure accelerates the consolidation of the clay layer in addition to groundwater depletion of the aquifer. Localized ground subsidence may also be caused by the occurrence of more than normal density of silt and fine sand lenses which serve as drainage paths accelerating consolidation in the soft clay. Another pattern usually occur on ground adjacent to canal excavations is subsidence due to slow lateral movement of the adjacent ground towards the canal. Furthermore, subsidence commonly occurs adjacent to underground drainage systems may be due to erosion of the subgrade sandy material due to leakage at cracks and joints in the drainage channels.

Evaluation of Subsidence Effects

Typical damages to land subsidence in AIT campus are found in many places

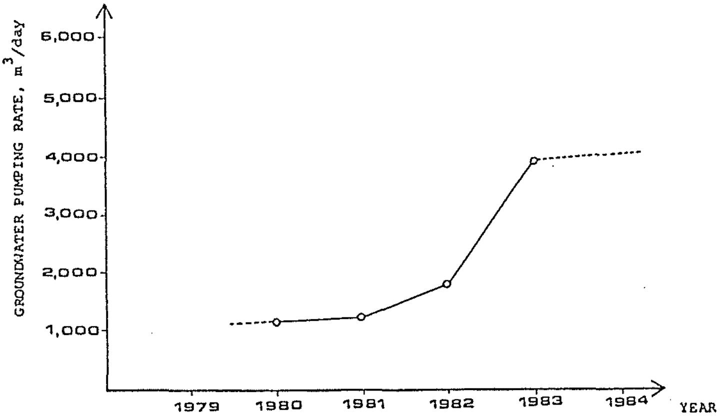

Fig. 2 Groundwater Pumping in AIT Campus

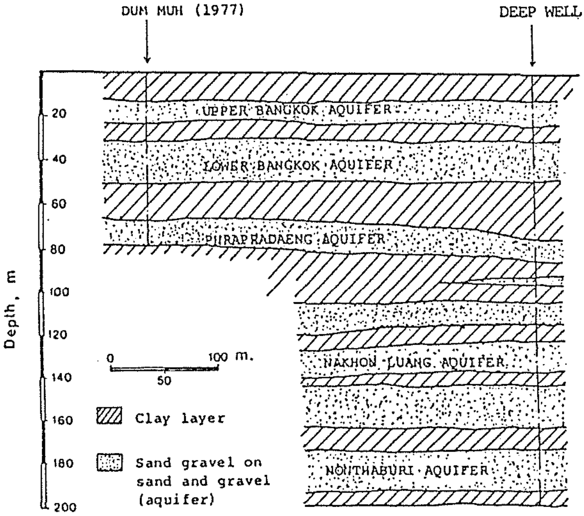

Fig. 3 Soil Profile East-West Section at AIT Campus

as indicated in Fig. 1 in the form of cracks, ground movements, and differential settlements. The subsidence observations were carried out by precision surface levelling with respect to a reference point, the deep compression indicator which is anchored down to 200 m depth. Two runs of precision levelling in interval of 3 months were carried out on the fixed steel rulers on buildings and walkways, surface settlement plates, pavement markers, and compression indicators. The locations of the observed points are indicated in Fig. 1.

Among the existing buildings, the worst damage which necessitated immediate repair occured in the Registry Office in the Administration Building. The floors and walls which directly rest on the ground surface and had not been connected to the tie beams have subsided, separated from the columns,

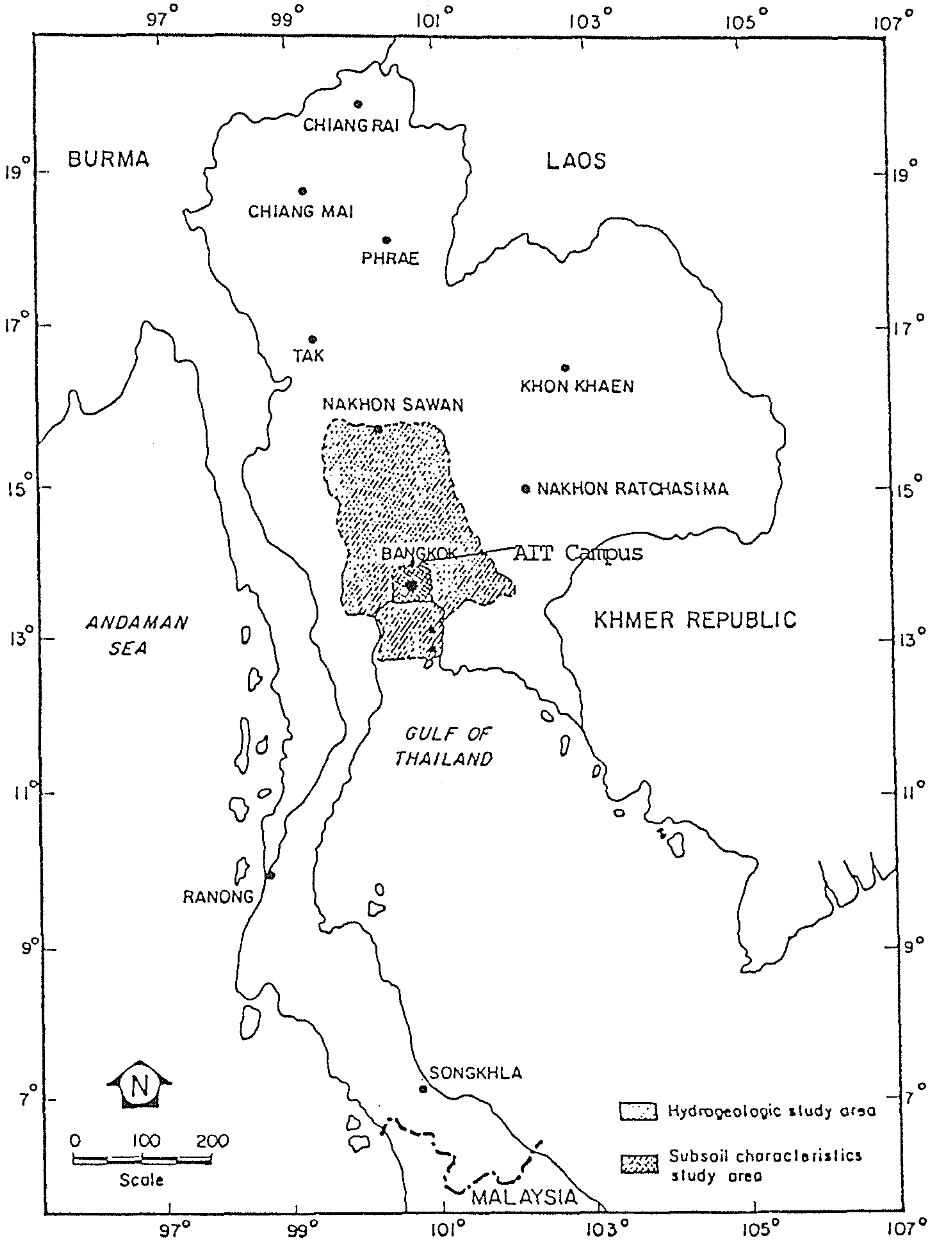

Fig. 4 Map of Thailand Showing AIT Campus and Areas of Previous Subsidence Investigation due to Well Pumping (After AIT, 1980).

and cracked. A detailed case study on the rehabilitation of the Registry Office was done by Bergado (1983a). In other parts of the Administration Building, differential settlements as plotted in Fig. 5 produced cracks in the first floor walls while the columns which are supported by pile foundation showed negligible settlements. Among the observed points as indicated in Fig. 1, a maximum settlement of 0.67 cm was measured in 3 months period.

The other damages can be seen in the Academic Buildings, namely: SEC and GTE Building and HSD and ENV Building. Both buildings show cracks in the first floor walls (Fig. 10a, b) due to differential settlements with a maximum of 2.40 cm in 3 months as plotted in Fig. 6 and Fig. 7 on observed points indicated in Fig. 1. The horizontal and vertical cracks are evident in the weakest portion of the walls such as lines of mortar joints. Measured settlements of the columns on piles were negligible.

The walkway between the Hockey and Football Fields (see Fig. 1) registered differential settlements as plotted in Fig. 8. A maximum settlement of 4.30 cm was measured in 3 months period. The larger settlement registered in this area may also be caused by erosion of the sand subgrade below the walkway due to rain water run-off.

The NZ Faculty Housing Area also suffers structural damages due to subsidence of the surrounding ground relative to the houses which are supported by piles. The sidewalks are cracked forming scarps around the houses (Fig. 11a, b) and the servant's quarters which directly rest on the ground are sinking with a maximum measured settlement of 3.94 cm in 3 months period which occur in House No. 1 (point 1C). Some of the settlements are plotted in Fig. 9a and 9b on points indicated in Fig. 1. Most of these damages occur in the houses adjacent to the canal.

In the asphaltic pavements, longitudinal cracks are found near the side slopes. The cracks of the concrete pavements in the tennis courts are found in random pattern. BM1 to BM12 whose locations are shown in Fig. 1 were used as observation points on the pavement (nail on pavement) around the campus. By interpolation, the Central AIT campus pavements have average settlement of about 0.5 cm in 3 months period.

In open fields, localized depressions can be observed in Football and Hockey Fields, Golf Course, and surrounding areas in Student Villages. The resulting subsidence measured in 5 surface settlement plates has an average of about 2.0 cm in 3 months period. It is also observed that the settlement of the point near the pumping well as indicated in Fig. 1 registered higher settlement.

Compression of the Soil Layer

Soil layer compressions were observed from shallow compression indicators at 10 m and 20 m depths and deep compression indicator at 200 m depth. Located near the AIT campus entrance, these instruments were installed during previous research work (AIT, 1978). The results of the recent measurements are tabulated in Table 1. In the period of 3 months, the observations indicated a compression of 2.28 cm in the top 10 m (soft clay layer) and a compression of 2.34 cm in the top 20 m. Considerable compression occured in the soft clay layer. The compressions in the upper 10 m and in the upper 20 m of subsoil are more or less similar in magnitudes. This large compression in the upper soft clay layer may cause the large differential settlements occurring in the AIT campus. The magnitude of subsidence below 20 m down to 200 m depth was relatively small. The measured compression was 0.19 cm in 3 months period.

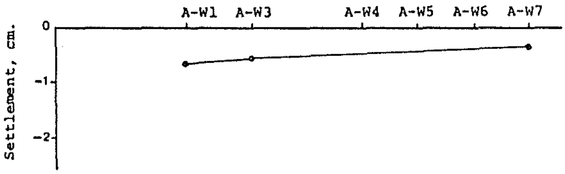

Fig. 5 Settlement of the First Floor Walls, Administration Building (See Fig. 1)

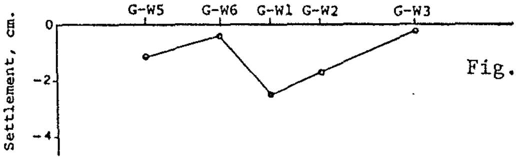

Fig. 6 Settlement of the First Floor Walls, GTE & SEC Building (See Fig. 1).

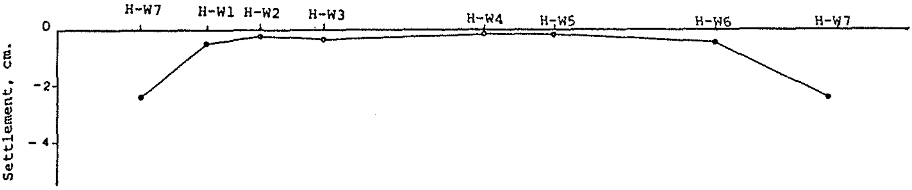

Fig. 7 Settlement of The First Floor Walls, HSD & ENV Building (See Fig. 1)

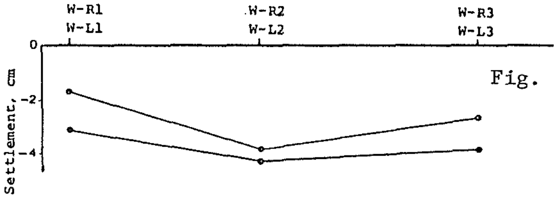

Fig. 8 Settlement of the Walkway Between Hockey and Football Fields

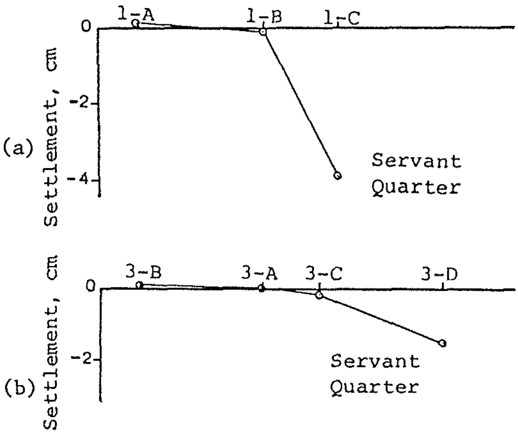

Fig. 9 Settlement in NZ Housing Area
a) House No. 1
b) House No. 3

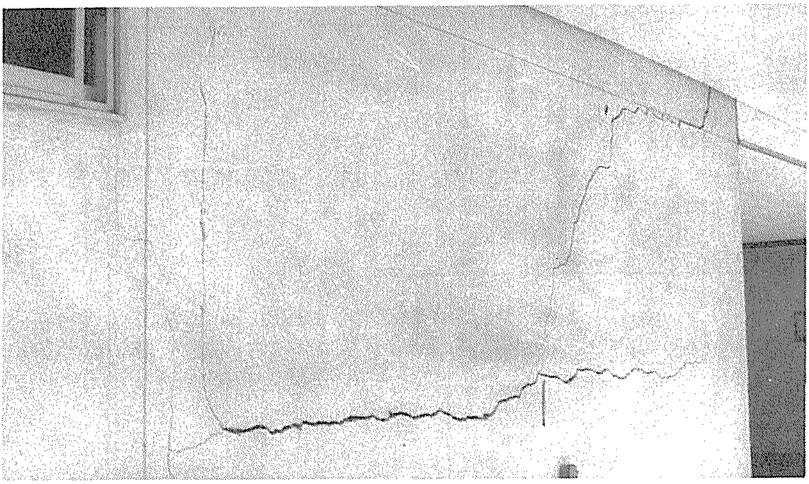

Fig. 10a Cracks in the First Floor Walls in ENV and HSD Building

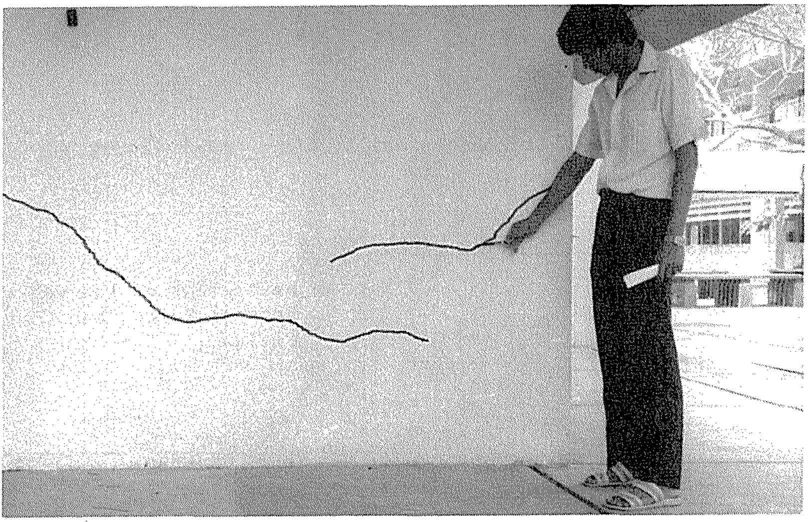

Fig. 10b Cracks in the First Floor Walls in SEC and GTE Building

Monitored Subsidence Correlated with Subsoil Conditions

Table 2 lists the locations, settlements, and types of damages. It can be seen that the area where damages occur registered large settlements.

Based on the subsoil vane shear strength profiles (Fig. 12 and Fig. 13) on line C and Line D as indicated in Fig. 1, the areas of large subsidence coincide with weaker subsoil. The vane shear strength profiles are more variable and the presence of a weaker layer pockets (shear strength $\leq$ 2.5 t/m^2) within the soft clay layer is more pronounced (Fig. 15). Line C extends from the NZ Housing Area through the Football and Hockey Fields to

Fig. 11a Damage of Sidewalk at NZ Housing No. 1

Fig. 11b Damages of Sidewalk at NZ Housing No. 7

the Academic Buildings. Line D pass through the NZ Housing Area through the Registry Office in the Administration Building to the Academic Buildings. All of these areas have evidences of large subsidence either structural failure or ground depressions. The existence of weaker layers in the soft clay are also confirmed from the results of the pressuremeter tests. The areas where evidences of subsidence occur coincide with the presence of thick, compressible layers with pressuremeter modulus, Em, below 25 kg/cm^2. Figure 14 shows the profile of Em values on line P1 indicated in Fig. 1. The damages are listed in Table 2.

Table 1 The Summary Results of Compression of Soil Layer

Compression Indicator	Period (months)	Depth Interval (m.)	Compression (mm.)
CI-1 to CI-2	3	1-10	22.80
CI-1 to BM	3	1-20	23.40
CI-1 to CI-4	3	1-200	25.29
CI-2 to BM	3	10-20	0.60
CI-2 to CI-4	3	10-200	2.49
BM to CI-4	3	20-200	1.89

Table 2 Settlement and Damages of Structures, AIT Campus

Location (refer to Fig. 1)	Type of Structure	Settlement in 3 months (mm.)	Type of Damage*
Walkway			
WR1-WL1	Column	15.45	a,b
WR2-WL2		5.45	a
WR3-WL3		12.43	a,b
ENV & HSD			
HW1	1st floor wall	5.46	c
HW2		2.42	c
HW6		4.45	c
HW7		24.15	c,d
GTE & SEC			
GW1	1st floor wall	24.67	c,d
GW2		11.39	c
GW3		3.39	c
GW5		11.93	c
Admin			
AW1	1st floor wall	6.86	c
AW5		5.54	c
AW7		3.90	c,d
NZ Housing	Servant Quarter wall		
1-C		39.44	e
3-D		11.49	e
7-B		7.73	e

* Note: a = Cracks in roof beam
b = Cracks in column
c = Cracks in the first floor wall
d = Separation between wall and ceiling
e = Separation between column or wall and pavement.

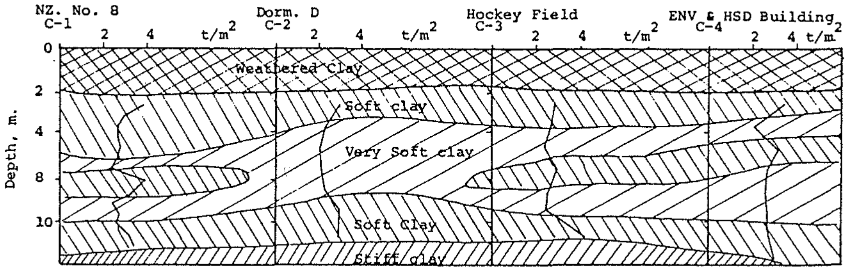

Fig. 12 Vane Shear Strength Subsoil Profile Along Line C (see Fig. 1)

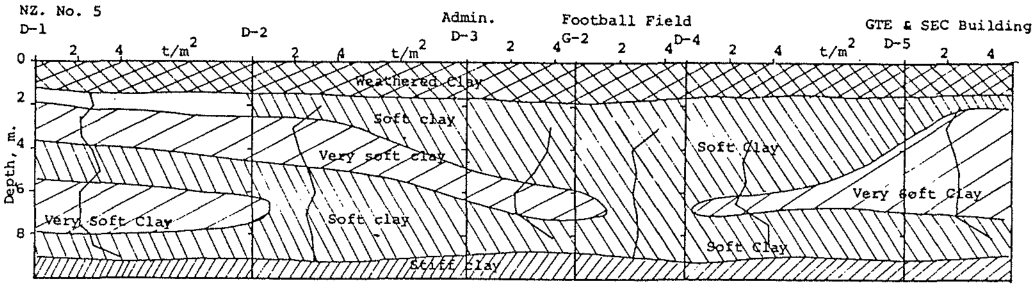

Fig. 13 Vane Shear Strength Subsoil Profile Along Line D (see Fig. 1)

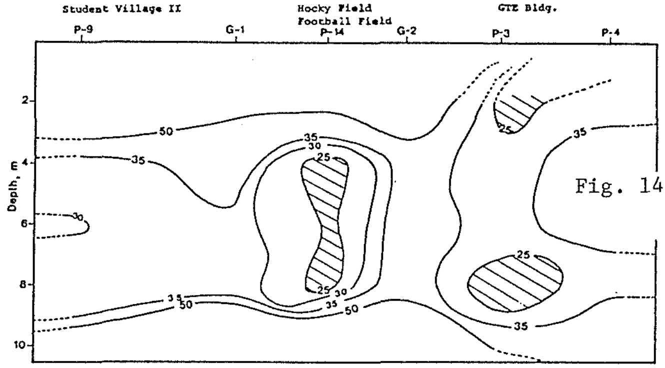

Fig. 14 Profile of Pressuremeter Modulus, Em in the section A1-E5, AIT Campus (Line P1)

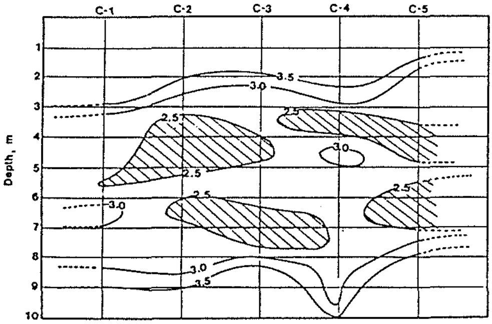

Fig. 15 Profile of Undrained Shear Strength along Section C1-C5 from Field Vane Shear Tests (Line C)

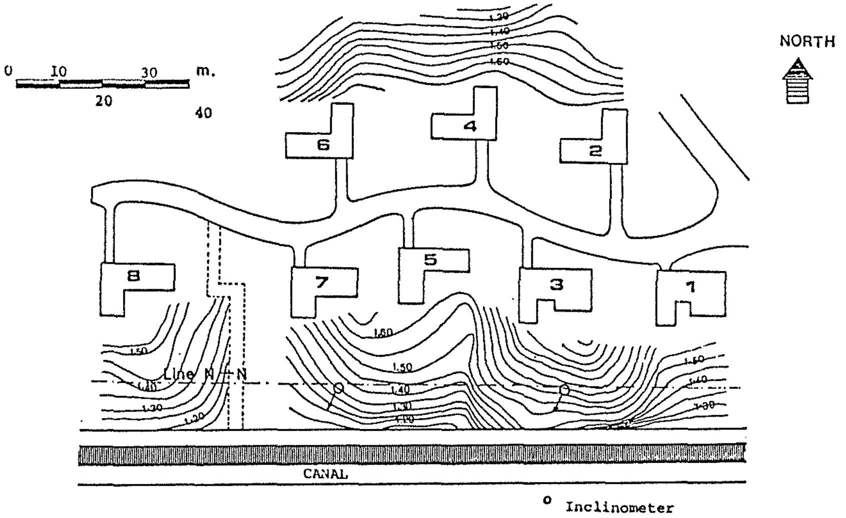

Fig. 16 Contour Elevation at New Zealand Housing, AIT Campus

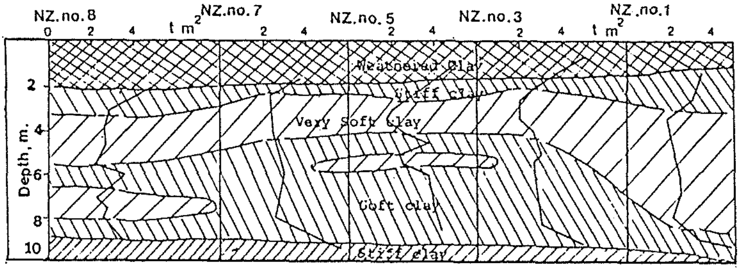

Fig. 17 Soil Profile along Line N-N at New Zealand Housing, AIT Campus

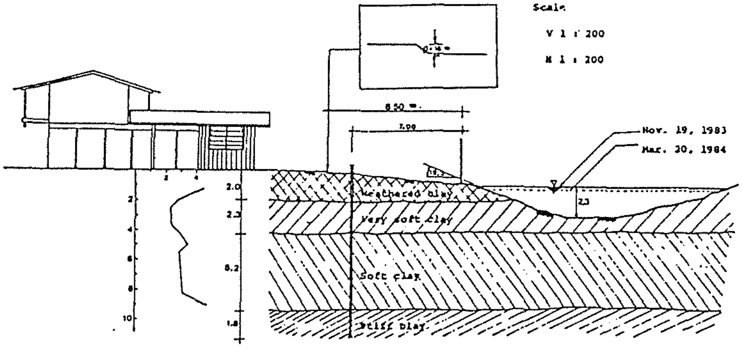

Fig. 18 Soil Profile and Inclinometer Location at NZ Housing, No. 3

Localized ground depressions occurring in open fields and highly loaded areas may also be caused by the presence of silt and fine sand lenses known to exist in the Rangsit soft clay that may serve as drainage paths accelerating subsidence due to well pumping from the leaky aquifer. Figures 12 and 13 show complicated shear strength profile than other areas in AIT campus. May be the density of silts and fine sand lenses in these areas is more than the others.

However, not all settlements are due to the weak subsoil and well pumping. Settlements are also caused by the erosion of the sand fill beneath pavements in areas near drainage trenches and underground sewers. The erosion occur in the subgrade through cracks in the concrete and through construction joints.

The subsidence in the NZ Housing Area as discussed previously may be caused mainly by slow lateral movements of the ground towards the adjacent canal. In three months period, the horizontal ground movements measured from inclinometers in the vicinity of House No. 3 and No. 7 were 1.13 cm and 0.55 cm, respectively, with locations and resultant directions indicated in Fig. 16. Soil profile through line N-N (Fig. 17) shows weaker layer with undrained vane shear strengths ≤ 2.5 t/m^2 in the soft clay at depths of 3 to 4 m. As shown in Fig. 18, the loss of lateral support as a result of the canal excavation may caused the slow lateral creep of the ground due to either the relative displacement of the upper weathered crust over the weaker subsoil or the slow squeezing of the weaker layer underneath the weathered crust.

Conclusions

From the results of this study, the following conclusions can be drawn.

In three months period, the average ground surface settlement is about 2.0 cm with maximum values near the pumping well. The average settlement on the pavements in Central AIT Campus is about 0.50 cm in 3 months period.

At the NZ Housing Area, a maximum settlement of 3.94 cm (in 3 months) was measured in the servant's quarters which unlike the houses are not on piles. Maximum settlements in three months period of 2.5 cm, 2.4 cm and 0.69 cm were measured in the damaged walls of the SEC and GTE building, HSD and ENV building, and Academic building, respectively. The walkway between the Hockey and Football Fields registered a maximum settlement of 4.39 cm in 3 months period.

Compressions of the subsoil showed that in the top 10 m and 20 m of the subsoil, compressions of 2.28 cm and 2.34 cm were measured, respectively, while compressions of the deep layers were relatively small.

The evidences of subsidence in the form of damages to existing structures and depressions of the surrounding ground can be correlated with the presence of low strength and very highly compressible zone in the soft clay underneath their corresponding locations. This large settlements in the damaged areas may also be caused by the higher than normal presence of silt and fine sand lenses that may serve as drainage paths resulting in an accelerated consolidation of the soft clay. However, settlements of pavements may also be due to the erosion of the sandfill in the subgrade.

The all around subsidence of the surrounding ground in the NZ Housing Area is mainly caused by either a slow horizontal movements of the surface hard crust over the very soft clay towards the canal or the slow squeezing of the soft clay towards the canal.

The present rate of groundwater pumpage is 4000 m^3/day. The well pumping for water supply in AIT campus is done in the Nonthaburi Aquifer at 180-200 m depth.

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LAND SUBSIDENCE IN CHINA

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Abstract

The most serious land subsidence occurs in areas with thick and fine-grained loose sediments or with shallow-buried karst, which is mainly attributed to groundwater pumping for water supply or dewatering of the mine. Besides, it is also closely related to the local geological environments. The paper describes the engineering geological and hydrogeological conditions and their influence on land subsidence in some of the studied areas, and gives a brief account of the measures to bring land subsidence under control.

General

According to the statistics since the founding of the People's Republic of China,most of the events of land subsidence or collapse caused by man's activities took place in the plain areas with thick and fine-grained loose Quaternary sediments because of overextraction of groundwater or in karst areas because of water supply or mine draining.

Land subsidence in plain areas caused by overextraction of groundwater

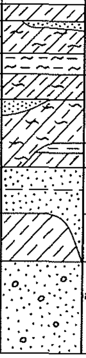
Land subsidence in plain areas caused by overextraction of groundwater generally occurs in the delta region along the coastal plain in the eastern part of China and in the urban areas of the alluvial plains along the big rivers, such as Shanghai, Wuxi, Changzhou, Tianjin, etc. In these areas, the Quaternary sediments are characterized in common by great thickness and fine grain size, though they are quite different in their origin and distribution. The geological structures are normally complex, with alternating multilayered clayey soil and confined water-bearing formations.

Taking Shanghai as an example, the authors describe the geological conditions of the front range of the delta region of the Yangtze River where the city of Shanghai is situated, to elucidate land subsidence in the plain areas of East China.

I. Engineering geological characteristics of the Quaternary sediments in the urban area of Shanghai

The Quaternary sediments in the urban area of Shanghai has a thickness of about over 300m. Above the depth of 150m up to the surface are interbeds of clayey soil and silt and fine sand of littoral, neritic and fluvial facies, while below are alternating fluvio-alluvial sand and lacustrine clayey soil. According to the engineering geological characteristics of the sediments, 13 layers can be divided from upper to lower, including eight clayey soil layers, one watertable aquifer (the same layer of surficial top soil) and five confined aquifers. Below the second confined aquifer are three stiff clay layers, with void ratio all less than 0.7, and compressibility coeffecient a_{1-2} all less than $0.025 cm^2/kg$. The test and measurement results show that the compression amount each year is considerably low. Above the second confined aquifer, i.e., above the depth ranging from 70m up to the surface, there exist one confined aquifer and five clayey soil layers, including the surficial soil layer, three weak clayey soil layers (the compressive layers) and one dark green stiff clay layer (Table 1).

Table 1. Major physico-mechanical property indices of the soil layers above the 2nd confined aquifer

Geological time		Depth (m)	Thickness (m)	Columnar section	Engng. geol. layer	Engng. geol. sublayer	Physico-mechanical proterties					
							Water content W %	Volume weight γ g/cm³	Void ratio ε	Liquid limit Wl %	Plastic limit Wp %	Compressibility coefficient a1−2 cm²/kg
Q_4	Q_4^4	0−4	0−4		Top soil	1−1	35.0	1.86	0.98	35.9	21.3	0.039
		3−15	0−12			1−2	35.3	1.83	1.00			0.025
	Q_4^3	3−15	3−12		1st compres.layer	2−1	43.3	1.77	1.21	33.9	20.8	0.077
		10−20	3−10			2−2	48.4	1.72	1.37	42.3	22.9	0.100
	Q_4^2	20−30	3−10		2nd compres. layer	3−1	34.3	1.83	1.00	34.6	20.8	0.042
	Q_4^1	25−35	0−20			3−2	31.9	1.84	0.94	32.8	19.1	0.043
						3−3	33.1	1.83	0.98	33.5	20.5	0.042
Q_3	Q_3^3	25−45	0−7		Stiff soil	4	24.6	1.99	0.71	33.4	18.7	0.023
		30−40	0−7		1st aquifer	5−1	28.2	1.92	0.80	27.2	16.9	0.015
	Q_3^2	35−45	0−10			5−2	26.2	1.92	0.77			0.017
		45−75	0−25		3rd compres. layer	6−1	34.4	1.84	0.99	35.8	20.2	0.042
						6−2	28.0	1.92	0.81	28.5	17.7	0.029
		70−100	20−30		2nd aquifer	7						

1-clayey soil plus sand; 2-muddy clayey soil; 3-muddy clay; 4-stiff clay; 5-fine sand or silt and fine sand; 6-sand plus gravels.

The three compressive layers have a high water content, and a void ratio approximately 1 or >1, indicating high or medium-high compressibility. Among them, the 1st compressive layer has the poorest engineering geological properties, its overconsolidation ratio being 1-1.2. The 2nd layer is characterized by its increasing thickness with the decreasing thickness of the underlying dark green stiff clay layer; and the 3rd layer is usually composed of alternating muddy clayey soil and thin beds of silt and fine sand. The deformation of these three compressive layers is the main cause leading to the land subsidence in Shanghai. The stiff clay layer of dark green colour under the 2nd compressive layer is 0-7m thick. It has a very dense structure, with an overconsolidation ratio ranging from 2.1 to 2.2. Under the action of the additional stress induced by the drawdown of groundwater level caused by pumping, it exhibits very slight deformation and even can dispel some of the action of the stress to reduce the compression volume of its upper layer. Therefore the existence of the dark green layer has a direct influence on the land subsidence of the studied area.

II. Relationship between land subsidence and geological structures

It is known that the phenomena of land subsidence in Shanghai was first observed early in 1921. The accumulative settling amount during the period from 1921 to 1949 was 639mm, averaging 29mm/year. Since 1949, land subsidence has become serious with the yearly increasing exploitation of groundwater for the development of economic construction. An extensive subsiding area has been gradually formed, involving the Yangpu district in the eastern part and the Jingan, Putuo and Changning districts in the western part, where groundwater extraction has been mostly concentrated. Till 1965, the central area of subsidence had a maximum accumulative amount of 2630mm. It has been proved through geological investigation and long-term observation that the main cause of land subsidence is the compression of soil layers induced by overextraction of groundwater. Besides, geological structures of the soil layers have also a bearing on land subsidence. Based on the conditions of the three compressive layers above the depth of 70m and the existence of the stiff dark green clay layer, and based on their different combination with the 1st and 2nd aquifers, four engineering geological structure areas can be classified in Shanghai as shown in Fig. 1.

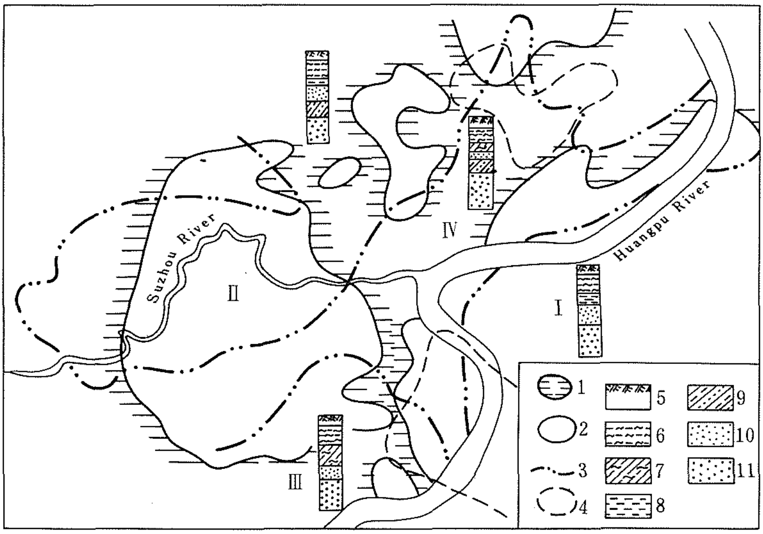


Fig. 1. Schematic map showing the relations between the geological structures and land subsidence in the urban area of Shanghai
1-subsiding zone; 2-zero line of subsidence, 1965-1976; 3-engineering geological structure area; 4-hydraulic connection area for aquifers I and II.

The 1st structure area is constituted by the 1st compressive layer, the dark green stiff clay layer and the 1st and

2nd aquifers. As there exist very thick water-bearing sand layers, and a very thin compressive layer, particularly the dark green clay layer, the land surface has rebounded considerably after the measures was adopted.

The 2nd structure area is constituted by the 1st and 3rd compressive layers, the dark green stiff clay layer and the 1st and 2nd aquifers. Though the accumulative subsiding amount was even higher than that of the 1st structure area before the treatment because the 3rd compressive layer was thicker, the land surface rebounded quite well.

The 3rd structure area is constituted by the 1st and 2nd compressive layers and the 1st and 2nd aquifers. It had a fairly large accumulative subsiding amount before the measures was adopted because of the absence of the dark green stiff clay layer and the hydraulic connection between the 1st and 2nd aquifers. After the measures being adopted only the subsiding rate was reduced but the 1st and 2nd compressive layers still remained under a slightly compressional condition.

The 4th structure area penetrating the central part of the urban area in NE-SW direction is constituted by the 1st, 2nd and 3rd compressive layers and the 1st and 2nd aquifers, with the absence of the dark green stiff clay layer. As there exists a greater number of compressive layers of great thickness than in the other areas, this area had the highest accumulative amount of subsidence before the measures were taken, its engineering geological condition being the poorest. After the measures, the 3rd compressive layer was basically prevented from compaction, but the 1st and 2nd compressive layers remained under the state of compaction and deformation to a slight extent.

Summing up the above, the relations between the characteristics of the engineering geological structures and land subsidence can be summarized as follow:

1. The compression rate of the 1st and 2nd compressive layers depends upon whether there exists a hydraulic connection between the 1st and 2nd aquifers. Actually the compression rate of the 1st and 2nd compressive layers in the 3rd structure area where a close hydraulic connection exists between the 1st and 2nd aquifers is proved to be higher than that in the 2nd structure area where no hydraulic connection exists.

2. The compaction of the 1st compressive layer has a close relation to the existence or absence of the dark green stiff clay layer. The 1st compressive layer displays a greater compaction in the 3rd and 4th structure areas with no dark green stiff clay layer.

3. The microdeformation of the 1st and 2nd compressive layers at the present moment is mainly dependent upon the rheologic characteristics and the thickness of the soil layer.

III. Measures taken to prevent land subsidence

According to the principle of consolidation, the static water level of the aquifer will draw down when groundwater is overextracted, thus leading to the dewatering and compaction of the soil mass caused by the declining porewater pressure and the increasing intergranular stress. However, under artificial groundwater recharge, the porewater pressure increases when the water level recovers and the soil mass rebounds when the intergranular stress reduces. The geologists of the Shanghai Geological Department have studied the mechanism of land subsidence and the relationship between the geological condition and land subsidence and adopted certain methods to recover and lift the groundwater level in order to bring the land subsidence under control. Good results have been achieved. The measures include:

1. Rational use of water and reasonable groundwater extraction

Since 1963, it is stipulated that groundwater can only be pumped in summer by industrial departments, factories and textile mills which use groundwater for cooling purposes. With the decrease of pumping, the groundwater level has recovered gradually year after year. As a result, the rate of land subsidence has correspondingly slowed down. For instance, the groundwater extraction in 1965 was only 42% that in 1963, in the mean time the subsiding amount decreased from 79.9mm in 1963 to 22.1mm in 1965.

2. Artificial recharge

As groundwater is usually pumped in summer, deep wells are often used in winter for groundwater recharge. Cold tap water (surface water) is used for the injection to speed up the recovery of groundwater level and on the other hand to store cooling water for summer use (Fig. 2).

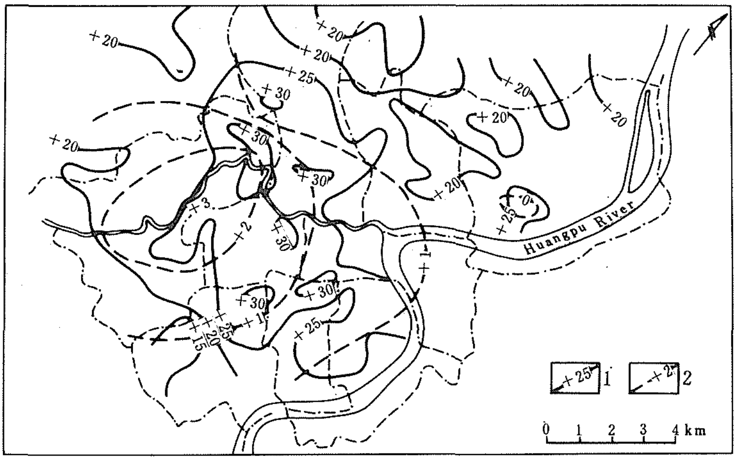


Fig. 2. Relation between groundwater level and land subsidence (rebound period, 1975-1976)
1-isoline of ground surface rebound during the period from September, 1975, to March, 1976 (mm);
2-peak piezometric isoline of the 2nd aquifer, 1976 (m).

3. Pumping readjustment of exploitable layers

The 4th and 5th water-bearing layers in Shanghai are the least extracted, because of their great depths and warm water temperature (24-26°C), which is unsuitable for cooling. Since 1968, artificial recharge for the 4th and 5th aquifers using the low-temperature tap water has been taken to increase the pumping amount of the two in summer and correspondingly to decrease the extraction of the 2nd and 3rd aquifers. Consequently the land subsidence has been alleviated and brought under control to a certain extent.

Ground surface collapse in shallow-buried karst area

Carbonate rocks are widely distributed in China, the outcropped and semioutcropped covering some 1.20 million square kilometres of the total territory of China. The carbonate rocks in some of the areas had once exposed over the ground surface to corrosion, thus being intensely karstified. In those shallow-buried karst areas, the thickness of the loose Quaternary sediments still governed by the climatic and hydrogeological conditions is generally less than 30m. Ground surface collapse caused by water gushing or draining in mines or by water supply in urban areas are quite common there, for instance in Hunan, Guangdong and Guangxi in South China, and in Shandong, Liaoning and Anhui in North China.

I. Case histories of karst collapse

1. Land collapse in Fankou lead-zinc mine, Guangdong

The Fankou mine is situated on the northern margin of the Dongtang basin. It is surrounded by low mountains and hills. The Quaternary sediments within the basin have a thickness of 10-30m, composed of clayey soil and sandy soil intercalated with sand and gravels, and underlain by middle-upper Carboniferous limestone averagely about 150m on top of the ore body. Karst are more developed in the upper part of the limestone, in which two water-bearing zones can be distinguished. The upper one of the two is about 70m thick, the karstified portion occupying 4.5%, where karst caves generally have a diameter of 0.5-3m, the maximum reaching 20m; the lower one is a weakly karstified zone with a karstified portion of 0.39%. In the karst water-bearing zones in a confined state, the groundwater level is at an elevation of approximately 100m. In order to ensure the safety of mining, water must be drained beforehand. Consequently, a cone of water table drawdown of about 13 million square metres came into being after the dewatering of the mine in the period from 1965 to 1977. The center of the depression cone of drawdown reaching 100m at an elevation of zero. The total drainage of eleven years reaches 140 million cubic metres, accompanied by various kinds of collapses accounting for 1400 in number, and covering an area of 4.90 million square metres (Fig. 3). The collapsed body may be 1-5m both in diameter and depth, the largest having a diameter of 40m and a depth of 30m or more. The scope and depth of the collapse has been increasing closely with the expanding of the cone of depression. The collapse generally occurred at the sections where underground karst developed, at the central area of the cone of depression, and along the fault, the beach or in the water-logging depressions.

2. Land collapse in Tai'an city, Shangdong

The Tai'an basin lies in the low hilly area, its elevation being ±135m. The Quaternary sediments generally has a thickness of 20-30m. The top part of the sediments is composed

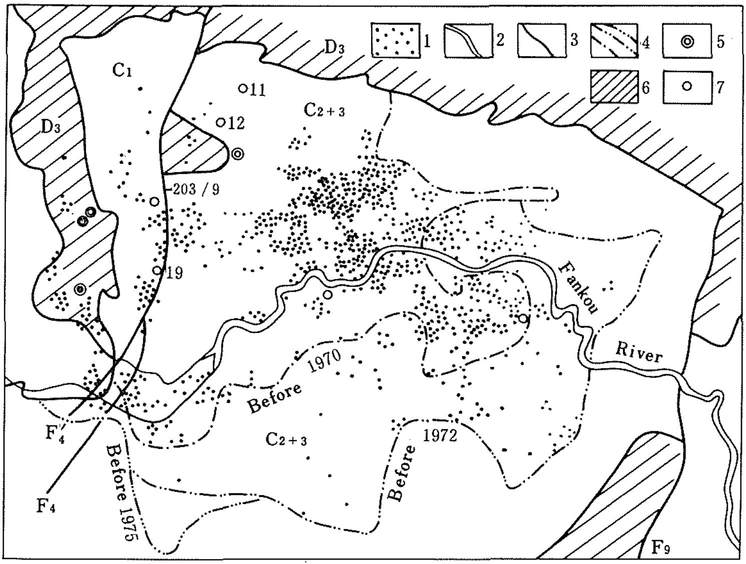


Fig. 3. A plane schematic map showing the yearly development of land collapse along the river banks in lowland after the dewatering of the Fankou mine
1-collapse pits; 2-river; 3-fault; 4-boundaries of yearly collapse scope; 5-mine shaft; 6-Devonian confining bed; 7-borehole.

of brownish yellow clayey soil 5-10m thick; next is fine to medium grained sand plus gravels 2-3m thick, whick is the major watertable aquifer; the third layer is seriously weathered gravel and soil layer; the fourth clayey soil layer containing rock fragments. The bedrock underlying the Quaternary is Upper Cambrian limestone striking NW and dipping NE with a dipping angle of 10-20 degree. In the rock mass fissures and fractures are well developed in northwest and northeast directions, dipping sharply. Karst is developed intensely. According to the statistics from 230 boreholes, up tp 91% karst caves or cavities were encountered. More than 5% of the sections are corroded mostly in the rock mass at the level over an elevation of 90m. The karst caves are generally of fissure type, with a height of more than 10m and width of less than 10m. The water-bearing limestone formations are usually under a confined state, the water level is close to the water table of the unconfined aquifer in the Quaternary sediments. Since 1970, the highest extraction of groundwater has surpassed three times that of the exploitable, leading to a sharp drawdown of groundwater level from the original elevation of 127m to below the surface of the bedrock at 93m (Fig. 4). The land collapse often happened when the groundwater level went down below the surface of the bedrock, and expanded succesively to dozens of places.

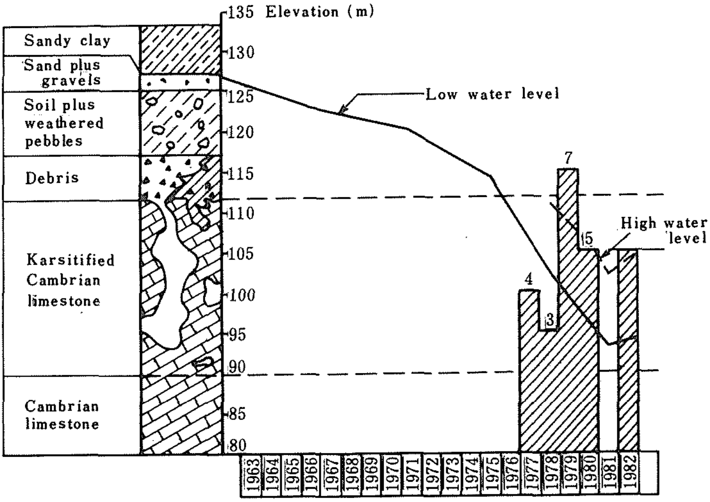


Fig. 4. Correlation between groundwater level, number of collapse , and lithologic characters in Tai'an area

3. Land collapse at the boundary of Yulin, Guangxi
The boundary of Yulin lies in the Lunggang valley, the bottom bedrock of which is middle Devonian limestone overlain by the Quaternary sediments of less than 10m thick. The upper part of the Quaternary is clayey soil and the lower part is occassionally sand and gravels, intercalated sometimes with soft muddy soil. Since the groundwater level lowered down sharply during the arid autumn of 1980, large-scale land collapse happened in the night of the 4th of January, 1981, with a total number of 400 or more big and small, the influence area reaching one square kilometer. The biggest has a size 120m long, 70m wide and 52m deep.

II. Distribution and origin of land collapse

1. Distribution of land collapse is related to the local geology and geomorphology and hydrogeology, mostly in intensely karstified areas with morphological combination patterns such as hilly depressions, depressions with ridges, valleys with peaks or plains with residual peaks. The overlying Quaternary sediments usually is less than 30m, composed mainly of clayey soil or clayey soil in the upper part and sandy soil or sand and gravels in the lower.

In the collapse areas the karstified aquifers often or seasonally remain in a confined state. The collapse usually occurred under the condition when the groundwater level was fluctuating around the lower limit of the covering bed or when it went down below the surface of the bedrock.

Collapse also tend to occur along the fissure belts, or at the belts of concentrating joints and their intersecting points, i.e., the corroded caves, channels, etc.

The collapses are concentrated near the central areas of the cone of depression or the main channel of groundwater flow.

2. Preliminary analysis of the origin of land collapse

The formation mechanism of the land collapse is complicated. In terms of the forming rate and different origins of the collapses, two types can be recognized: collapse caused by vacuum suction, and that by scouring. The former usually occurs in shallow karst water-bearing formations which are covered by compact soil bed under tightly closed condition. When groundwater level goes down rapidly the originally water-filled caves or cavities become relatively vacuum and produce suction or negative pressure, and simultaneously the top covering bed loses the underneath support by the water head pressure. As a result, the permeating pressure increases and the self weight of the soil layers as well. When the combination force of the components of self weight of the soil bed, the permeating pressure and the suction force becomes greater than the shear strength of the covering bed, land collapse will occur suddenly as the first case described above. The latter appears where groundwater level fluctuates at the lower limit of the overlying soil bed and scouring expands the cavities and finally leads to land collapse. This can be exemplified by the case of the collapse at the Yulin boundary in Guangxi region.

3. Prevention of land collapse

In order to control or ease land collapse, the connecting passages between the karst caves have been surveyed by means of comprehensive exploration methods. Land collapse forecasting map has been compiled for working out appropriate water discharge scheme and relevant prevention measures.

Rational groundwater withdrawal is determined according to the exploitable resources in the urban and mining areas for the purpose of water supply so as to keep the water level under a confined state. When pumping or draining is arranged, holes are drilled to let the air in to fill the corroded space, thus vacuum or suction will not be formed.

During pumping or discharge, effective measures are taken to prevent the surface water flow into the collapsed areas. For instance, diversion of surface water or changing local water ways of the waterlogged lowland to prevent reverse flow of surface water and the development of scouring and collapse of soil layers. Pits and depressions caused by land collapse are back filled.

Conclusion

1. Many factors may induce land subsidence or collapse. In China, groundwater discharge or draining in plains or shallow-buried karst areas, leading to a water table drawdown, may considered as the main cause. However, other factors are also nonnegligible.

2. The phenomena of land subsidence in Shanghai reflect the common characteristics of that in urban areas in coastal plains of China, such as Wuxi, Changzhou, Ningbo, Tianjin, etc. The deformation of co pressive soil layers are closely related to not only their engineering geological properties and preconsolidation state but also the geologic structures. It has been proved that land subsidence has been brought under control to a better extent by means of restricting groundwater withdrawal, artificial recharge and readjusting the pumping layers of the exploitable aquifers.

3. In shallow-buried karst areas, the soil cover is not very thick. Therefore the carbonate rocks underneath tend to be subject to intense corrosion, which provides the conditions for land collapse. It is often due to the combined

actions of scouring and vacuum suction caused by the lowering down of the groundwater level when the aquifer is changed from confined to unconfined state. Based on geological conditions and karst features in such areas, some measures can be taken , such as proper groundwater extraction and discharge, appropriate exploitation, and drilling of air inlet hole, diversion of surface water flow, etc. Other treatments such as backfilling of the collapsed pits are also beneficial for easing the consequences of land collapse.

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Note: The references are all in Chinese. The latter two unpublished.

OVER RECHARGE EXPLORATION OF GROUNDWATER AND LAND SUBSIDENCE IN TAIYUAN CITY, SHANXI PROVINCE, CHINA

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Abstract

Land subsidence was found in Taiyuan, an industrial city in Shanxi plateau of North China, since the early 1970s after precise leveling. The maximum subsidence was up to 1.232m in 1982. The configuration of subsidence isoline conformed basically with the isopotential line of groundwater drawdown. The subsidence centre situated at the front verge of Taiyuan alluvial fan where cohesive soil interlayer became more and thicker.

By the history and existing situation of ground water development in Taiyuan, associated with the hydrogeological condition, using mathematic model and records of long term observation, this paper proved that the land subsidence in Taiyuan was due to intensive development over recharge. This paper stressed that attention should be paid to keep the balance of yield and recharge when developing ground water from alluvial fan of an industrial city. Otherwise, land subsidence was liable to occur even in inner land far away from seashore.

I.

Taiyuan is a city of heavy industry in North China. The ground water resources there were relatively abundant. At its north is the Shan Lan spring with a flow of 0.55m^3/s. At its south is the 'Never Old Spring' of the famous scenic spot -- Jin Ci Temple, with a flow of 2m^3/s. The water had been used for agricultural irrigation since the time of War State (2000B.C.) for the production of well known Jin Ci Temple rice. After the 1950s the extraction of ground water increased rapidly as a result of development in production and prosperity in population. Take the pumpage in 1950 as 1, then in 1960 it increased to 19.7, in 1970, up to 32, in 1978, up to 48.7 and in 1982 up to 100. Hence the ground water level of Taiyuan was lowered continuously as a whole and formed a cone of depression with an area of 290km^2(Fig.1). The centre of the cone was at the west of the city. The depression was as deep as 50-60 m. In 1979-1980 precise leveling of second and third order were conducted twice by Taiyuan Institute of Building Design and Municipal Surveying Team and resurveyed in 1982. An isoline map of land subsidence was produced (Fig.2) which showed that the maximum subsidence was 1232mm. It occurred in Wu Jia Pu at the south of the city. Area with subsidence over 1000mm was 7.1km^2 and that with subsidence deeper than 200mm was 245km^2. The area of drawdown cone conformed basically with that of land subsidence. The centres of drawdown and land subsidence depicted that land subsidence was correlated with ground water exploitation. The centres were not coincided with each other.

There was a distance about 5.5km between these two centres, which showed moreover that land subsidence was related to the variation of alluvium distribution. Analyses of these two aspects are emphasized in the following paragraphs.

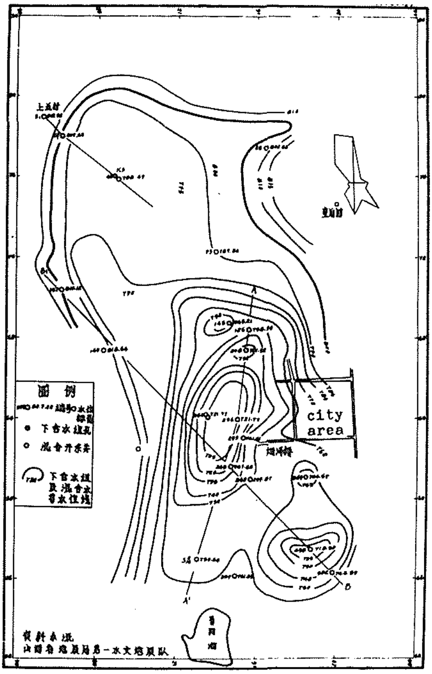
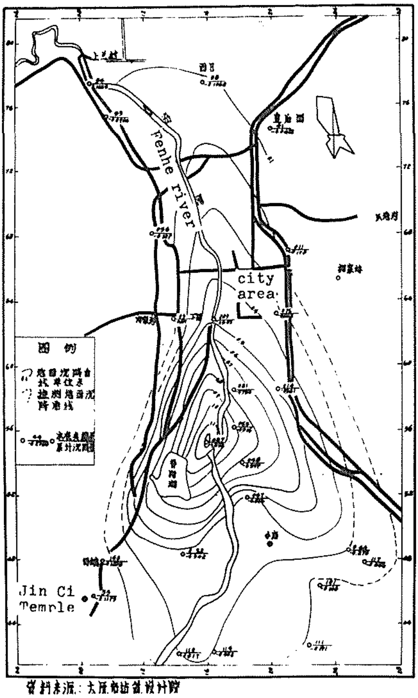


Fig.1 The equipotential lines of ground water



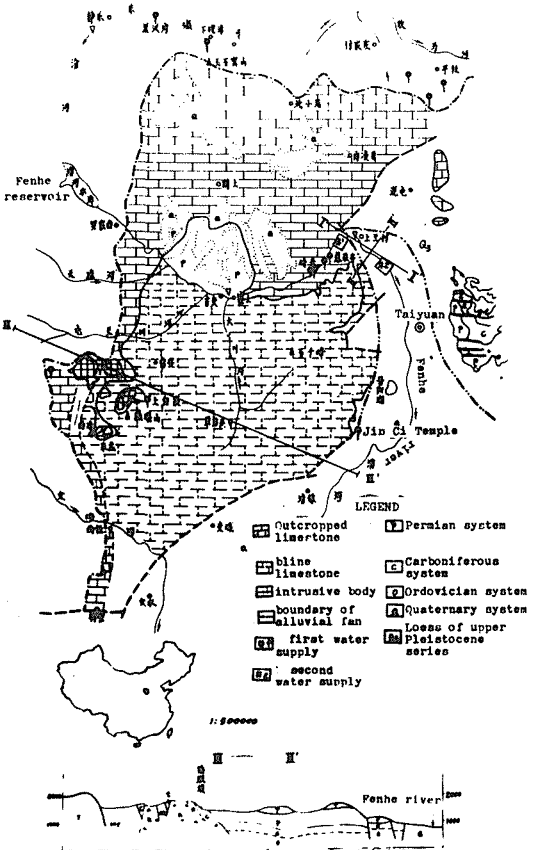
資料来源：太原市勘测设计院

Fig.2 Iso-subsidence lines

II.

Taiyuan is located at the north end of central Shanxi basin extending in the direction of north to south. The total length of the basin is more than 100km and the maximum width in the central part is 25km. The depression is more than 1000m in depth. An alluvial fan of $29x10km^2$ in area and 300m in thickness has been deposited at the gorge as the Fenhe river flowed from the West Mountain of Taiyuan into the basin. Taiyuan city lies on the central part of the fan (Fig.3).

The length of Fenhe river up to Taiyuan city is 196km, covering an area of $7640km^2$. The average flow for many years was $16.16m^3/s$. The maximum flow was $1480m^3/s$ during the rainy season of July, August and September, then the river water was muddy with sandy particles up to $0.524ton/m^3$. From December to January and February of the next year it is an arid period, the water then is very clear and being drained from ground water. Great quantity of river water seeps into underground when Fenhe river flows through the exposed Ordovician limestone stratum with an area about $1200km^2$ of West Mountain, where the river becomes a dry valley after the rainy season. But spring water gushes from both sides of the limestone nearby the gorge (see Fig.3), then water appears again in the river bed and the flow increases along its path. The flow reaches its maximum amount, 9m^3/s, at the gorge. After the construction of reservoir at the upstream of Taiyuan in 1964, the flow reduced to 5.5m^3/s. In 1965 the first water supply yielding 2.5m^3/s was built in the river bed of Fenhe river gorge for which shallow wells were drilled in gravel layer (not more than 40m in thickness) of the river bed. The flow reduced further to 3m^3/s, which fully denoted that the flow of clear water in Fenhe river came from the drainage of karst water.



This drawing is excerpted and being modified from "Report on Ground Water Research of West Mountain, Taiyuan." contributed by Institute of Earth Science

Fig.3
The situation of Taiyuan City and its geological circumstance

In 1972 the second water supply was built, that means additional 40 deep wells were drilled about 10km downstream of the first water supply, or at the head of Taiyuan alluvial fan which was more than 150m in thickness. Its capacity was 2.6m^3/s. The yield was more than 5000m^3/day by single artesian well. The author had taken part in the verification of ground water resources before the pipeline was laid. According both to profiles (Fig.4,5) provided by hydrogeological investigation and the existence and decrease of clear water flow of Fenhe river, the relation between the recharge of ground water in alluvial fan and the drainage of West Mountain karst water was recommended as expressed in Fig.6. After being recharged with the river seepage the West Mountain karst water drained downstream along the valley and was blocked by the gravel layer of alluvial fan (its permeability was far less than that of limestone at the gorge). Most of the karst water drained upwards towards the valley and became clear water flow of Fenhe river, with only a small portion of it flowed further on to recharge the alluvial fan. This inference could be drawn from three facts as follows:

(1) After the accomplishment of the first water supply, an observation hole No.k, of 230m in depth was drilled 206m into the limestone. The water level in limestone raised

with the increase of depth (Table 1). That showed the karst water had an upward component of flow velocity.

(2) As the water table and flow were steady, cone of drawdown did not expand and its influenced extent was not more than 500m in the first water supply, which indicated that the recharge source was nearby and the amount of recharged water was greater than the yield of $2.5m^3/s$.

(3) Pumping tests of nine wells conducted during the exploration of the second water supply indicated that the total flow was only $34000m^3/day$. But the water level was unsteady throughout the pumping period of one month and the cone of drawdown extended over 10km.

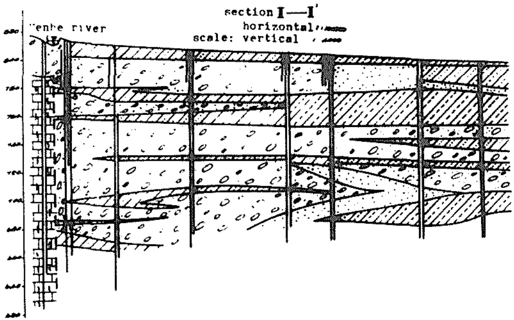


Fig.4 The longitudinal profile of the head of alluvial fan

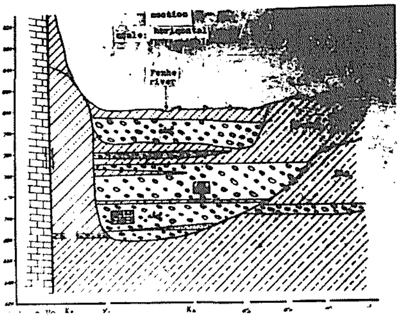


Fig.5 The transverse profile of the head of alluvial fan

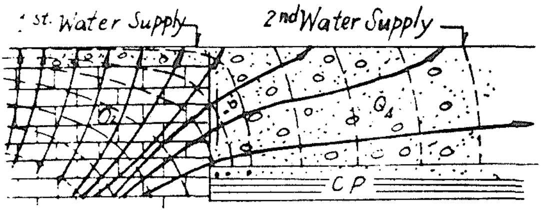


Fig.6 The scheme of relation between recharge of the alluvial fan and drainage of karst water

Table 1

borehole depth (m)	26	48	79	120	180
water level (m) taking the ground surface as zero	-0.56	+0.8	+1.03	+1.15	+1.26

The amount of recharge to alluvial fan from karst water was determined as less than 34000m /day. As the recharge to alluvial fan was so small that the author proposed to call off the construction of the second water supply but to enlarge the first water supply instead. However, this proposal was not accepted at that time. Now, after ten years, not only the second water supply was put in operation but also thousand wells were drilled in the urban district and south suburbs. The incessant over extraction caused water table drawdown of the whole alluvial fan.

III.

In order to confirm that the lowering of water level was the result of extraction over recharge the following mathematical model was recommended: assume the aquifer was in the shape of rectangle and its length and width were L and B respectively. The aquifers were simplified as a single layer, homogeneous and isotropic.

The recharge end at the north was maintained constant flow and the drainage end at the south was kept constant water level. Both laterals of aquifer were impervious. There were N_1 wells in the aquifer to be pumped with constant flow Q, then problem (I) is given as follows:

$$
(\mathrm{I})\begin{cases}
\dfrac{\partial^2 h}{\partial x^2}+\dfrac{\partial^2 h}{\partial y^2}=\dfrac{S}{T}\dfrac{\partial h}{\partial t} \\
t=0\ ; \quad h=h_0 x/L \\
x=0\ ; \quad h=0 \\
x=L\ ; \quad \dfrac{\partial h}{\partial x}=-h_0/L \\
y=0 \quad \dfrac{\partial h}{\partial x}=0 \\
y=B \quad \dfrac{\partial h}{\partial y}=0 \\
x=x_i\ , \quad y=y_i\ ; \quad \int_0^{2\pi}\dfrac{\partial h}{\partial r}\Big|_{r=r_w\to 0}
\end{cases}
$$

$$d\theta=\frac{Q_i}{2\pi r_w T}\ ; \quad i=1,2,3\cdots N_1\ , \quad r=\sqrt{x^2+y^2}$$

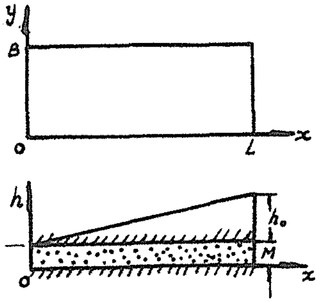


Fig.7 The scheme of the mathematical model

where h is water head
L is the length of aquifer
B is the width of aquifer
T is transmissivity
S is storage coefficient
r is radius of well
suffix i is the serial number of well
x,y are coordinates
t is time

Solution of problem (I) is:

$$
\begin{aligned}
h=h_0 x/L-\frac{4}{\pi^2 TB}\sum_{i=1}^{N_1}Q_i\Big\{L\sum_{n=1}^{\infty}\frac{1}{(2n-1)^2}\big(1-e^{-T(2n-1)^2\pi^2 t/4L^2S}\big) \\
\times\cos\frac{(2n-1)\pi(L-x_i)}{2L}\cdot\cos\frac{(2n-1)\pi(L-x)}{2L} \\
+\frac{1}{L}\sum_{n=1}^{\infty}\sum_{m=1}^{\infty}\frac{1}{(\frac{2n-1}{2L})^2+(\frac{m}{B})^2}\big(1-e^{-(T/S)((\frac{2n-1}{2L})^2+(\frac{m}{B})^2\pi^2 t}\big) \\
\times\cos\frac{(2n-1)\pi(L-x_i)}{2L}\cdot\cos\frac{n\pi y}{B}\cdot\cos\frac{(2n-1)\pi(L-x)}{2L}\cdot\cos\frac{m\pi y}{B}\Big\}
\end{aligned}
\qquad (1)
$$

The flow at the drainage surface (x=0) of aquifer is

$$Q=T\int_0^B\frac{\partial h}{\partial x}\Big|_{x=0}\cdot dy \qquad (2)$$

Differentiate h of (1) for x and substitute it into the above equation, then integrate, get

$$Q=TB(h_0/L)-\sum_{i=1}^{N_1}Q_i \qquad (3)$$

The first item on the right side of (3) represents quantity of recharge Q_r . This equation illustrates that when $N_1 = 0$, or when no well is pumping, the flow of drainage of ground water

$$Q_d = TB(h_o/L) = Q_r \qquad (4)$$

When the discharge of pumping equals to that of recharge, or

$$TB(h_o/L) = \sum_{i=1}^{N_1} Q_i \qquad (5)$$

$$Q_d = 0$$

From equations (3),(4),(5) it is known that the reason why the water level of aquifer keeps steady during extraction is because the flow of extraction is less than that of recharge. It also proves that the flow of extraction is out of the flow of recharge and takes place of the flow of natural drainage. When the flow of extraction equals to that of recharge there will be no more natural drainage but the cone of drawdown will still be steady. If the flow of extraction increases, then unsteady drawdown would be induced as a result of the consumption of ground water storage. The unsteady drawdown "s" may be counted from the steady cone of h(x,y), the expression of which could be derived from (1) by letting $t \to \infty$. Then the problem of unsteady stage due to over recharge development should be modified as

$$\frac{\partial^2 s}{\partial x^2} + \frac{\partial^2 s}{\partial y^2} = \frac{s}{T}\frac{\partial s}{\partial t}$$

$$\begin{array}{ll}
t = 0 & s = 0 \\
x = 0 & \frac{\partial s}{\partial x} = 0 \\
x = L & \frac{\partial s}{\partial x} = 0 \\
y = 0 & \frac{\partial s}{\partial y} = 0 \\
y = B & \frac{\partial s}{\partial y} = 0
\end{array}$$

$$x = x_j \quad ; \quad y = y_j \quad ; \quad \int_0^{2\pi} \frac{\partial s}{\partial r}\Big|_{r=r_w} \cdot d\theta = \frac{Q_j}{2\pi r_w T} \quad ; \quad j = 1, 2, 3 \cdots N_2$$

where j is the serial number of well increased during over recharged extraction

Q_j is the flow of those wells

N_2 is the total number of wells

The solution of problem (II) is

$$S = \{(\sum_{j=1}^{N_2} Q_j)/LBS\}t + S_2(x, y, N_2, x_j, y_j, Q_j) \qquad (6)$$

where

$$\begin{aligned}
& S_2(x, y, N_2, x_j, y_j, Q_j) \\
& = \frac{2}{\pi^2 T}\sum_{j=1}^{N_2} Q_j \Big\{ \frac{B}{L}\sum_{m=1}^{\infty} \frac{1}{m_2}(1 - e^{-m^2\pi^2 Tt/B^2S})\cos\frac{m\pi y_j}{B} \cdot \cos\frac{m\pi y}{B} \\
& + \frac{L}{B}\sum_{n=1}^{\infty} \frac{1}{n^2}(1 - e^{-n^2\pi^2 Tt/L^2S})\cos\frac{n\pi x_j}{L}\cos\frac{n\pi x}{L} \\
& + \frac{2}{LB}\sum_{n=1}^{\infty}\sum_{m=1}^{\infty} \frac{1}{(\frac{n}{L})^2 + (\frac{m}{B})^2}\{1 - e^{-\pi^2 T((\frac{n}{L})^2 + (\frac{m}{B})^2)t}\}\cos\frac{n\pi x_i}{L} \\
& \times \cos\frac{m\pi y_i}{B}\cos\frac{n\pi x}{L}\cos\frac{m\pi y}{B} \Big\} \qquad (7)
\end{aligned}$$

As mentioned above the drawdown "s" in (6) is counted from the surface of steady cone. If "s" is counted from natural water level, then the drawdown of steady cone "s" should be added, thus

$$S = S_1(x, y, N_1; x_i, y_i, Q_i) + S_2(x, y; N_2, x_j, x_j, Q_j) + [(\sum_{j=1}^{N_2} Q_j)/\angle BS]t \qquad (8)$$

If drawdown of a certain point (such as a certain observation well) in aquifer is studied, then (x,y) is a fixed value, so the above equation could be expressed by a simple linear equation as

$$S = a + bt \qquad (9)$$

By (9) it is obvious that when the extraction is exceeded, any point in aquifer will have drawdown in linear relation with time and the slope of s-t line is in direct proportion to the yield of over recharge.

Fig.8 shows a set of s-t curves drawn with observation data of ten years (1971-1981) of three observation wells, wells k_2, k_3 were located at the second supply and another observation well k_1 was in limestone at the first water supply situated in the valley of Fenhe river but out of the head of fan. From these curves we learned that the water level declined in observation wells k_2 and k_3 while the water level in well k_1 kept steady. That shows the alluvial fan was under the condition of over recharged yield. For example, in well k_2, the slope "b" of curve s-t could be classified into three stages and over recharged yield could be calculated approximately according to

$$b = (\sum_{j=1}^{N_2} Q_j)/\angle BS$$

that is

$$1971 \sim 1974;\quad b = 1.6^{m}/3\,years = 0.533\,m/year;\quad Q_{over} = 42300\ T/day$$
$$1974 \sim 1979;\quad b = 10.7^{m}/6\,years = 1.783\,m/year;\quad Q_{over} = 142000\ T/day$$
$$1979 \sim 1981;\quad b = 4.3^{m}/1\,year = 4.3\,m/year;\quad Q_{over} = 341600\ T/day$$

Hence it is proved that the direct cause of cone of drawdown in alluvial fan is over recharged yield.

IV.

The coincidence of area of the extent of land subsidence and cone of depression reflected the relation of cause and effect between the two. The fact that the centre of drawdown was not coincide with the centre of land subsidence was related to the formation of alluvial fan. The head of alluvial fan was formed by sandy gravel layer as shown in Fig.r. At the centre and tail of the fan the sand layer became finer and thinner and the interlayer of cohesive soil increased and became thicker. As counted from the geological log of 196m deep well in the centre of land subsidence the cohesive layer was 64%. The compressibility of cohesive soil was greater than that of sandy gravel layer. But further to the south, though the proportion of cohesive soil would become greater the drawdown would become less and so the land subsidence would decrease gradually.

With the consolidation equation expressed by compressibility strain proposed by a Japanese, Oaku Mura (1967)

$$\frac{\partial^2 \varepsilon}{\partial z^2} = \frac{1}{C_v} \cdot \frac{\partial \varepsilon}{\partial t} \qquad (10)$$

where ε is the strain of cohesive soil layer produced by load increase caused by pressure decline

c is consolidation coefficient of soil; $c_v = k/m_v \gamma_w$

The permeability coefficient of cohesive soil layer in alluvial fan k=1-10^{-1} m/day, so the value of c_v should generally be 10^4 cm^2/yr.

Hence the initial and boundary condition of (10) could be written as:

$$\left.\begin{array}{ll} t = 0\;; & \varepsilon = 0 \\ z = 0 & \varepsilon = m_v p t / t_1 \\ Z = M & \varepsilon = m_v p t / t_1 \end{array}\right\} \qquad (11)$$

where m_v is the compressibility coefficient of soil layer
p is pressure stress
t is time interval between the start of loading and the accomplishment of calculation. Hence the range of t in the formula is $0 < t \leq t_1$

The solution of problem (10) and (11) is

$$\varepsilon = \frac{m_v p}{T_1}\left\{T - \frac{16}{\pi^3}\sum_{n=1}^{\infty}\frac{1}{(2n-1)^3}\left(1 - e^{-(2n-1)^2\pi^2 T/4}\right]\sin\frac{(2n-1)\pi z}{M}\right\} \qquad (12)$$

where $T = 4C_v t/M^2 \;,\quad T_1 = 4C_v t_1/M^2$ (13)

Compressible value of cohesive soil layer

$$S = \int_0^M \varepsilon\, dz$$

Substitute (12) into the above equation and integrate, then

$$S = \frac{M m_v p}{T_1}\left\{T - \left(\frac{8}{19} - \frac{9}{\pi^2}\sum_{n=1}^{\infty}\frac{1}{(2n-1)^4}\, e^{-(2n-1)^2\pi^2 T/4}\right)\right\} \qquad (14)$$

Finally the degree of consolidation of cohesive soil layer could be derived,

$$U = \frac{S}{M m_v p} \qquad (15)$$

The denominator of the above equation represents the final amount of subsidence.

Up to now, Taiyuan has not studied land subsidence systematically. There was neither case history of variation of subsidence with time nor essential information necessary for research work, such as the strutum structure of alluvial fan. Therefore it was difficult to check the amount of subsidence with (14). In order to confirm the cause of subsidence from the geological log of borehole at the centre of subsidence, we selected arbitrarily the 14th layer of clayey silt, 4.94m thick and study its consolidation coefficient and degree of consolidation. Hence we must determine m_v first. It is assumed that the maximum total amount of subsidence 1.232m was the result of compressing cohesive soil layers with the compressibility of sand layer being neglected when the drawdown was 59.3m (5.93kg/cm^2), then the average compressibility coefficient was

$$m_v = \frac{1.232}{112.25 \times 5.93} = 0.0018\,\mathrm{cm^2/kg}$$

which was used for the 14th clayey silt instead of its real value.

Take the c_v value from such a wide range as $c_v = 10^3 - 10^5$ cm^2/yr

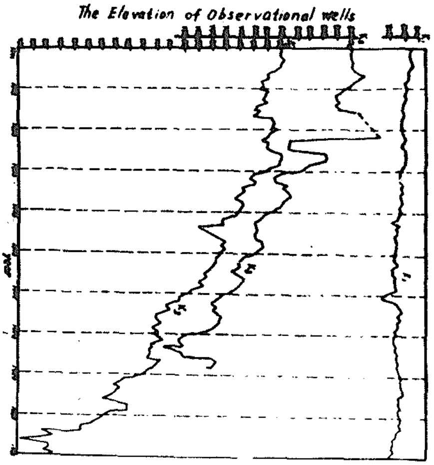


Fig.8 The drawdown-time curve of three observation wells

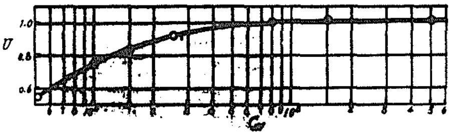


Fig.9 The calculated relation between the consolidation coefficient and degree of consolidation of the 14th layer clayey silt

and the results of corresponding degree of consolidation U calculated by (14) and (15) are shown in Fig.9.

The order of quantity of both the degree of consolidation U and the consolidation coefficient c_v was so reasonable that we may believe the maximum amount of subsidence was caused by drawdown.

Conclusions

It is proved by this paper that land subsidence may occur in alluvial fan of plateau basin under the condition of extensive drawdown caused by intensive development of groundwater, just like the case happened in silt layer along the seashore. Taiyuan is a proper example from which the following instructions could be drawn.

The ground water storage accumulated during the prolonged geological age in large alluvial fan should not be extracted at random only based upon its superficial phenomena, such as the high water head and the artesian wells. If extraction were really necessary the routine of recharge, flow, drainage should be investigated in anticipation and its recharge be accurately estimated. Gain and loss should be strictly considered based on prediction of the dynamic equilibrium of ground water after extraction. For example, the loss to tourist resources and other disadvantages due to the elimination of springs and scenic spots caused by drawdown and on the other hand, the benefit to agriculture brought about by the reclamation of saline land should be compared and then conclusion could be drawn.

If extraction were determined to carry out, attention should be paid to the following:

(1) The yield should be critically controlled below the existing recharge in order to prevent unsteady drawdown.

(2) Extracting site should be in the head of alluvial fan

where the aquifer is coarse grained, with less interlayer of cohesive soil.

(3) Do not extract from the tail of alluvial fan as the aquifer there is thin and the grains are fine. Not only the well yield is small but also large drawdown will be formed easily and compression of multi-layered cohesive soil will be induced so that land subsidence will occur. The example illustrated in this paper was an obvious one.

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MECHANISM OF LAND SUBSIDENCE AND DEFORMATION OF SOIL LAYERS IN SHANGHAI

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Abstract

This paper, based on an analytical study of sufficient data collected from in situ observations and laboratory experiments, endeavours to expound the mechanism of land subsidence and the rule of deformation of the soil layers in Shanghai. In the paper, particular mention is made of a close analysis of the unit deformation which reveals that the values of deformation of the soil layers can be readily evaluated by normalizing the observed deformation of different soil strata according to the range of variation of the ground water table and that the characteristics of the residual deformation of various soil strata can be observed in terms of the ratio between rebound and compression.

I. General Geological Condition of the Overburden

In Shanghai area, loose overburden deposited on the bed rock during the Quaternary period is as thick as 300 meters. It is divided into one phreatic water-bearing layer (unexploited layer), five confined aquifers (hereinafter called aquifers) and 13 engineering geological layers, as revealed by the analysis of the hydrogeological characteristics and the engineering-geological properties (see Fig. 1).

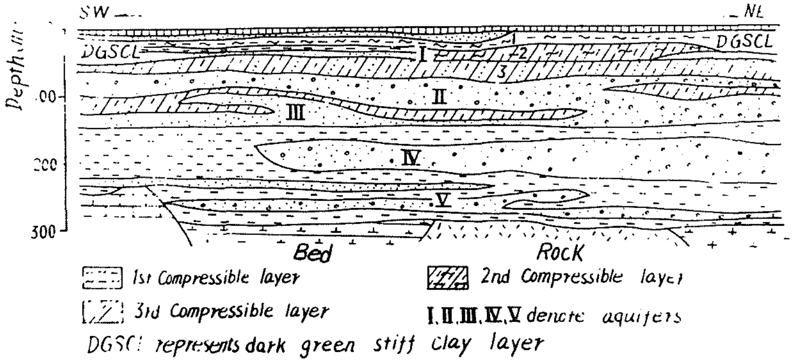

Fig. 1.—Hydrogeological profile of Shanghai urban area.

The data obtained from some deep bench marks indicate that the land subsidence mainly occurs in the upper portion of 70m thick of the three soft compressible layers. As shown in Fig. 2, the principal physical-mechanical indices of the three layers reveal that they will decrease as the layers increase. A detailed examination of the different combinations of the three

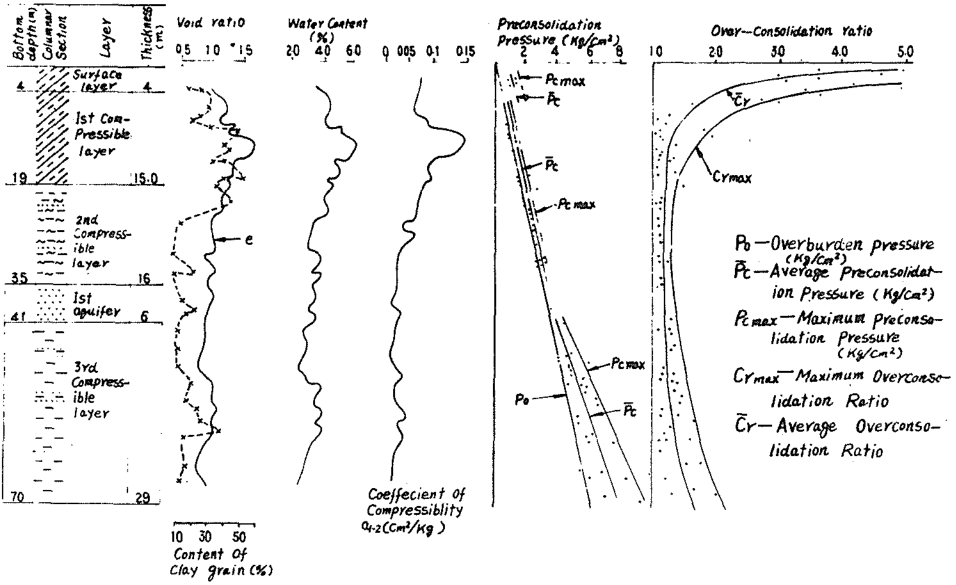

Fig. 2.—Variation of main physical-mechanical properties with the depth of three compressible layers above 70m in Shanghai urban area.

compressible layers and one dark green stiff clay layer, from the depth of 70m upward, and the hydraulic interconnection between the 1st and the 2nd aquifer discloses that the urban area of Shanghai may be divided into four geological structure zones of land subsidence (see Table 1).

II. The Basic Characteristics of the Deformation in Various Soil Layers under the Condition of Exploitation and Recharge of Ground Water Table

1. The 1st and 2nd Compressible Layers

The 1st and 2nd compressible layers (hereafter called the shallow soil) 3-40 meters below the ground surface, consist of clayey soil and littoral-epicontinental sea facies. The soil possesses soft engineering properties, a low permeability and a long time lag. Fig. 3 shows the variation of cumulative deformation and the fluctuation of water table in the shallow layers of the two different structure zones. The figure points out further that a remarkable amount of compression took place in the soil layers at a

Combined condition of Soil layers / No. of Zone	1st and the 2nd Compressible layer	Dark green Stiff Clay layer	1st aquifer	3rd Compressible layer	The hydraulic interconnection between the 1st and the 2nd aquifer
No. 1	Exist	Exist	Exist	Absent	With interconnection
No. 2	Exist	Exist	Exist	Exist	No interconnection
No. 3	Exist	Absent	Exist	Absent	With interconnection
No 4	Exist	Absent	Exist	Exist	No interconnection

Table 1.—The combined condition of various soil layers in the four geological zones.

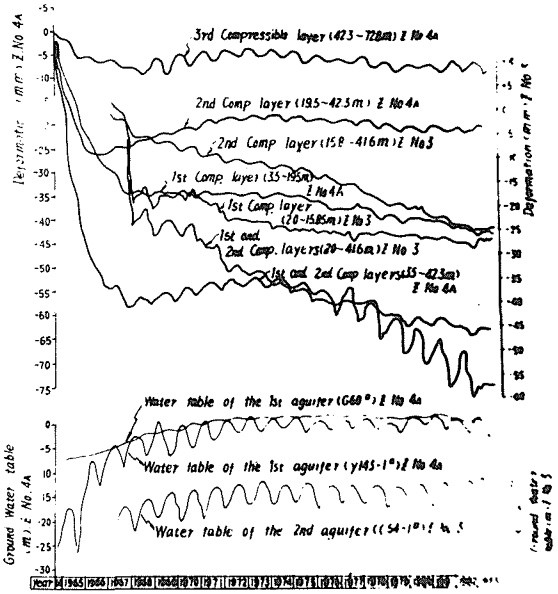

Fig. 3.—Curves showing the cumulative deformation of the 1st, 2nd, and 3rd compressible layers and water table fluctuation in the two different geological structure zones. (Zones No. 3 and No. 4A).

rate of 6-10mm/year prior to the proper precautions taken in 1966. However, the subsidence has begun to slow down since proper measures were adopted.

The laboratory rheological test proves that the shallow soil has a certain rheological behaviour. As seen in Fig. 4, the deformation of soil shows no tendency to approach stabilization during the 90 days of testing and the ratio between the secondary consolidation and the principal consolidation is either 2:1, or 3:1, when the load rate is 0.2 - 0.6 (Su, 1979). The rheological properties of the soil layers illustrate that a slight amount of compression at a rate of 1-2mm/year is still observed. Particularly, in the Zone No. 3 on account of the existence of the direct hydraulic interconnection between the 1st and 2nd aquifers, the cumulative compression is comparatively greater than those in the other zones. So the stabilization of the subsidence of the shallow soil becomes the key to the control of the land subsidence in Shanghai.

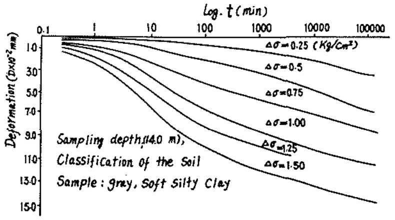

Fig. 4.—Curves showing the consolidated deformation (D) by odometer test (1st compressible layer) versus time (t).

2. The 3rd Compressible Layer

The deformation of the 3rd compressible layer involves two stages as shown in Fig. 3. The low water table stage prior to 1966 was regarded as the 1st stage, during which the amount of compression was fairly great because of the exploitation of the 2nd aquifer and the action of low water table on the layer resulted in a main compressed layer on land subsidence. Subsequent to that is the 2nd stage. During that stage, the recharge made the amplitude of ground water level rise, which would cause the layer to swell. And then the soil layer, along with the periodical rising and lowering of the water table, swelled and compressed alternately, thus resulting in a short time lag of consolidation and swelling effect. All this demonstrates that the deformation of the layer depended on the recent fluctuation of the water table resulting from the compression of the permeable consolidation, as the influence caused by the past fluctuation of water table had practically disappeared (Su, 1979 & Su, 1981).

The above rule of deformation can also be seen in Fig. 5, which shows that the slope of cyclic lines of stress-strain relation (tgαc) by pumping is much greater than that of the tgαs by recharging. But while the variation of stress occurs mainly in the proximity of the average preconsolidation pressure Pc value, the stress-strain relationship basically occurs in a linear pattern. This, therefore, explains that the elastic behaviour of the layer, at present, is gradually increased. Moreover, it can also be illustrated that the Pc value has an important effect on the 3rd compressible layer, and if the fluctuated amplitude of water table in the 2nd aquifer does not exceed its Pc value, the compression of the soil layer is only a pure residual settlement under the condition of repetitious loadings and unloadings.

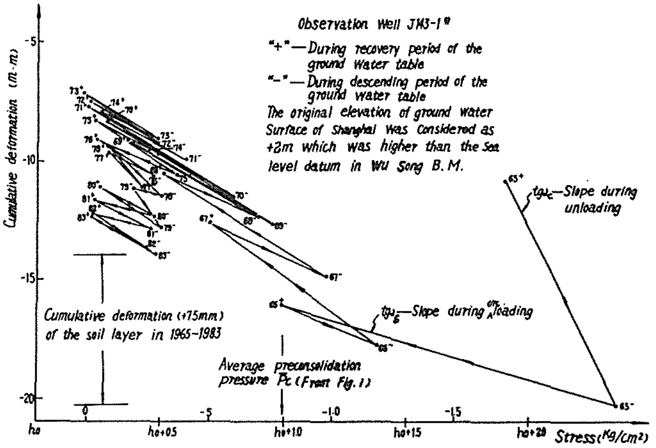

Fig. 5.—Stress-strain relation in the 3rd compressible layer (Zone No. 4A)

3. Sand Aquifers

The curves showing the variation of cumulative deformation and water table fluctuation in the 2nd, 3rd, 4th and 5th aquifers are shown in Fig. 6. It can be seen that, since the counter measures taken in 1966, the variation of deformation with time and the variation of water table with time in the above sand aquifers have been closely interrelated with both its peak and valley values corresponding with each other due to non-existence of time lag. Since then, the cumulative swelling of the 2nd and 3rd aquifers has reached 11.67mm and these values have helpfully stabilized the land subsidence in the urban area of Shanghai.

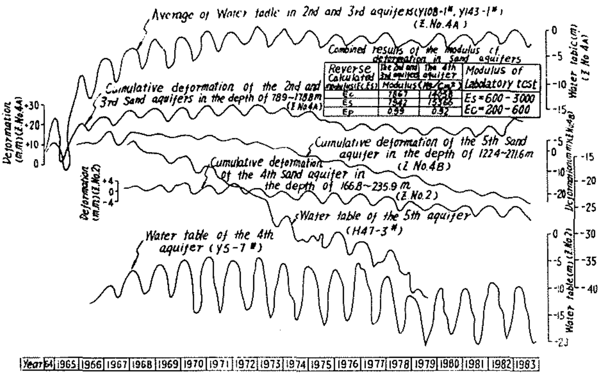

Fig. 6.--Curves showing the variation of cumulative deformation and water table fluctuation in the 2nd and 3rd aquifers (Zone No. 4A), 4th aquifer (Zone No. 2) and 5th aquifer (Zone No. 4B).

On the basis of a close analysis of the data obtained from the field observations, reverse calculation of the modulus of deformation Es (rebound) and the modulus of deformation Ec (Compaction) in the sand aquifers has been made (See Fig. 6).

As indicated, the Ec value may reach 7867 kg/cm^2 in the 2nd and 3rd aquifers and reach 14058kg/cm^2 in the 4th aquifer, while the Es value may be as high as 7942 kg/cm^2 in the 2nd and 3rd aquifers, and 15366 kg/cm^2 in the 4th aquifer, with a small difference between the two. This shows that the deformation of sand layers has approached the elastic state (Fig. 6). Therefore, if the water level drawdown can be raised to the beginning of elevation, most of the compaction in the sand aquifers will be rebounded.

Furthermore since 1968, a series of measures have been taken to control land subsidence on the upper portion of soil layers. One of them is regulating the sequence of exploited aquifers in the urban area. The rational amount exploited of the 4th and 5th aquifers is reasonably augmented and that of the 2nd and 3rd aquifers diminished. As a result, the elevation of water table in the 4th and 5th aquifers is gradually drawing down year by year (see Fig. 6). In the industrial area of the Western District of the city, where concentrated exploitation takes place in the 5th aquifer, the amount of compaction of the sand layer is relatively greater and consequently the sand layer becomes a main compressive one on land subsidence in this area (Fig. 6).

III. Mechanism of Deformation in Soil Layers under the Pumping-Recharging Action

The previous studies prove that the compression of soil layers caused by pumping is one-dimensional consolidation (Ren & Su, 1981, and Tsien & Gu, 1981). Drawdown of ground water level breaks equilibrium of pore water pressure distribution in original soil mass. As the pore water pressure distribution in original soil mass. As the pore water pressure decreases, the effective (intergranular) stress increases, thus bringing about the consolidation settlement of soil layers. Contrarily, on account of rechargir the water into the aquifer, the pore water pressure increases, the effective stress decreases, and the soil layers swell. These facts have been proved b field observation data of pore water pressure in Shanghai as illustrated in Fig. 7, which shows: that the pore water pressure varies with the depth of pumping-recharging aquifer, i.e., the nearer is the piezometer head to the aquifer, the greater pore water pressure will be, and vice versa. In the seasonal fluctuation of ground water level, the peak-valley value correspond with the water level of aquifer, pore water pressure and deformation of soil

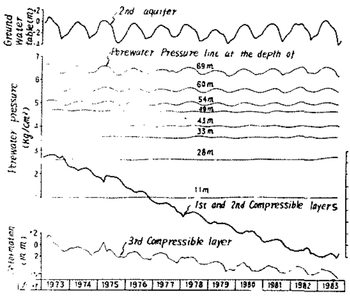

Fig. 7.—Time variation of the pore water pressure and the deformation of the layers (Zone No. 4c).

layer. The time lag is short. It shows that the change of pore water pressure is the main cause of deformation of soil layers.

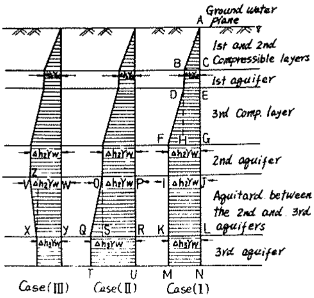

Fig. 8.—The distribution of effective stress increment under the action of different types of pumping.

As far as the mechanical effect is concerned, the increase of effective stress in soil layer caused by pumping (i.e. initial condition of consolidation) may be the combined result of the decreasing of buoyant support force and the action of seepage force (Fig. 8). The former is the decrease of pore water pressure (or hydraulic pressure) of the upper boundary in compressible layer as a result of pumping. The pressure in the upper part originally supported by the decreased pore water pressure, is now gradually transferred to the skeleton of the compressible layer and becomes the effective loading. The latter is the difference of the hydraulic head (i.e. the hydraulic gradient) at the top and the bottom of the compressible layer as a result of pumping. The seepage flow is hence induced. As the seepage flow acts directly on the skeleton, the layer is compressed. The seepage pressure is also known as dynamic hydraulic pressure. The time lag of compression varies with the permeability of the soil layers. When pore water pressure of each point in soil mass reaches a new equilibrium, the consolidation of soil will be completed. This consolidation process is generally followed by the classic consolidation equation (Ren & Su, 1981, Tsien & Gu, 1981), i.e.:

$$\frac{\partial u}{\partial t}=C_{VC}\frac{\partial^2 u}{\partial Z^2} \qquad \text{or} \qquad \frac{\partial u}{\partial t}=C_{VS}\frac{\partial^2 u}{\partial Z^2} \tag{1}$$

The buoyant force and seepage force mentioned above will act isolately or simultaneously on the compressible layers according to the various pumping conditions. The initial value ΔP can be expressed by formula (2) and (3) respectively, i.e. ΔP is the product or half-product of the drawdown (Δh) times the density of water (r_W). It may be expressed as follows:

$$\Delta p = \Delta h \cdot \gamma_w \qquad (2)$$

or, $$\Delta p = \frac{1}{2} \Delta h \cdot \gamma_w \qquad (3)$$

The three main pumping types of Shanghai district are shown in Fig. 8, which is used to explain the action of the two forces above. Let Δ hl, Δh_2, Δh_3, be the amplitude of the drawdown of the first, second, third aquifers respectively. Despite the forms of pumping, when the ground water level of the first aquifer draws down to the depth of Δh_1, the increment of the initial effective stress of the first and second compressible layers is expressed by triangular distribution (ABC). This is the action of seepage force. The direction of seepage flow is downward, and its value is shown by formula (3). The drawdown of the 1st aquifer will decrease the buoyant force of the upper layers by a value of $\Delta h_1 \cdot r_w$ equivalent to an external load acting on the 34d compressible layer. However, the drawdown of the second aquifer is greater than that of the first aquifer, i.e. $\Delta h_2 > \Delta h_1$, therefore, the stress simultaneously produces the same downward seepage force acting on the third compressible layer. The sum of these two forces results in increment of initial effective stress with regular trapezoid distribution (DEFG) in the soil layer. As for the aquitard between the second and third aquifers, the increment of initial stress varies with the drawdowns of the second and third aquifers. When $\Delta h_2 = \Delta h_3$, the effective stress is a rectangular distribution, such as IJKL in case I; when $\Delta h_3 > \Delta h_2$, the effective stress is a regular trapezoid distribution such as OPQR in case II; but when $\Delta h_2 > \Delta h_3$, the effective stress is a reverse trapezoid distribution as shown by VWXY in case III, the stress can be resolved into a rectangle (ZWXY) and a triangle (XVZ). Obviously, the former is the decrement of buoyant force and the latter the action of seepage force, the direction of which is upward. The seepage force is the negative value of the aquitard in relation to the buoyant force. Consequently, the seepage force is a mass force. It is directional, which may be either positive or negative. These are the fundamental differences between the consolidation by pumping and the ordinary consolidation by loading.

On the basis of the above formula (1), (2), (3), we can predict the land subsidence either by analysis method (Tsine & Gu, 1981) or by summation of layers method (Ren & Su, 1981).

IV. The Characteristics of Residual Deformation of Various Layers under the Cyclic Fluctuation of Water Table

At present, the land subsidence of Shanghai is the result of the combined actions of the compaction and swelling of various layers under the condition of seasonal ascending and descending of water table by pumping and recharging. Therefore, making a special study of the characteristics of residual deformation by pumping and recharging has a practical significance.

1. The Soil Tests of Cyclic Loading in the Laboratory

Some soil samples, taken from the depth of 70 meters upward, have been tested on the one-dimensional odometer with pumping and recharging device under the condition of cyclic loading in the laboratory. Use a water-tank 5 meters above the odometer to supply water, so as to simulate recharging of in-situ. A vacuum pump is used to produce vacuum pressure on the soil samples to imitate the actual drawdown of water table, as seen in Fig. 9. From the testing results as indicated in this figure, we can see that the residual deformation of soil is increased as the number of cycle (N) decreases. Although the number of cycle (N) has reached 60 times, the deformation of the soil sample has no tendency to approach stabilization at all.

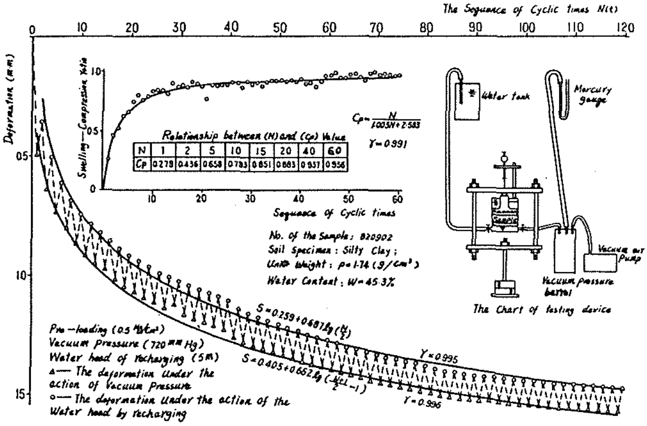

Fig. 9.—The testing results of the shallow soil on the one-dimensional odometer with a pumping and recharging device in the laboratory.

Its Cp value is equal to 0.95. It can be seen that at the same equivalent pressure difference of cyclic loading, the residual (unrecoverable) deformation can not be avoided.

2. The Characteristics of the Unit Deformation of the Soil Layers in Shanghai and the Study of the Characteristics of Residual Deformation

Unit deformation means the corresponding deformation of a layer caused by one meter of the variation of ground water table. It may be expressed as follows:

$$Is = \frac{\Delta Ss}{\Delta hs} \qquad (4)$$

$$Ic = \frac{\Delta Sc}{\Delta hc} \qquad (5)$$

Is, Ic are the unit deformation during the recharging and exploitation period respectively (mm/m); Δhs, Δh_C represent the rising and falling amplitude of ground water level (m); ΔS_S, ΔSc represent their corresponding deformation (mm).

Let $|I_S| / |I_C| = Cp$, which is called the swelling-compression ratio. When the Cp value is equal to 1, the residual deformation will be completed and the soil mass will reach the elastic stage.

By evaluation according to formulas (4) and (5), we have obtained the seven deep bench marks of the Cp values as drawn in Fig. 10. In this figure, we can see that the Cp value of the 1st and 2nd compressible layers have only reached 0.48–0.69. This explains that these soil layers have a longer residual deformation. In the 3rd compressible layer, or the 2nd 3rd aquifers, the Cp values have reached 0.85–0.93. This shows that the behaviour of the residual deformation is relatively small. Summing up the data of various soil layers, we obtained the surface B.M. of Cp value which is only equal to o.72. This demonstrates that the land subsidence of Shanghai, varying with the rising and descending of seasonal fluctuation of of water level, has an unelastic behaviour, i.e. when ground water level rises and drawdowns to complete a cycle, the elevation of land subsidence of Shanghai will be relatively decreased by 20 percent of the total deformation during this cyclic period.

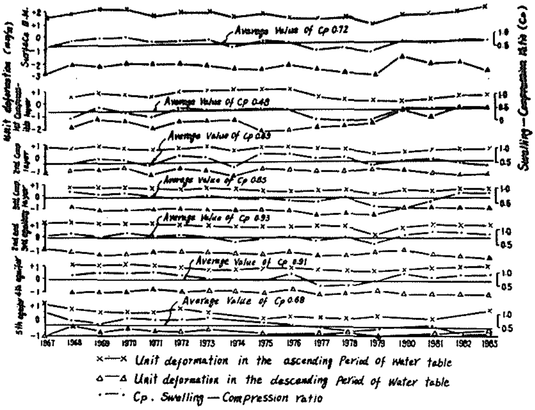

Fig. 10.—Time variation of unit deformation in various layers. (The unit deformation of the overburden as a whole is shown by surface bench marks).

For this reason, under the present condition of exploitation and recharging in Shanghai, we must make a further study on the controlling of the rising and lowering amplitude of water table and its variable types. By means of diminishing the amount of residual deformation as much as possible, we can further control the land subsidence in Shanghai.

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ANALYSIS OF THE CAUSE OF LAND SUBSIDENCE IN TIANJIN, CHINA

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Abstract

Based on the general geology of Tianjin area, the paper deals with land subsidence in the area, which may be attributed in the main to the overextraction of groundwater. However, strong earthquake and compaction of less consolidated are also considered non-negligible factors.

Introduction

Tianjin, which is one of the important industrial cities of China, is located along the coast of the Bohai Gulf west of the Pacific Ocean. It is about 120km northwest of Beijing. The flat relief of the Tianjin area where many depressions and lakes can be found has an elevation of only 3-5 meters, and is drained by the Haihe River pouring into the Bohai Sea. The loose sediments overlying the considerably fractured bedrock have a great thickness and quite complex hydrogeological conditions.

Since the year of 1959 when people became first aware of land subsidence, the maximum accumulative settlement of the ground surface reached 2.15m till 1982, the maximum subsiding rate was 261mm per year, and the subsided area was expanded to over 7300km^2. There exist three settling centres in the urban area, with an accumulative amount of subsidence more than 2m. (Fig. 1)

The effect of land subsidence imposed on the industry and agriculture and on people's life is an urgent problem to be solved.

Geology

I. The city of Tianjin is situated in the settling zone of North China Plain, the 2nd subsiding zone of the Neo-Cathaysian tectonic system. The urban area spans three structures of the system, i.e., the Jizhong depression, the Huanghua depression and the Cangxian uplift inbetween. Along the contact between the N-S uplift and the Jizhong depression on the western side and the Huanghua depression on the eastern side are the respective Hangou fracture and Cangdong fracture. Within the Cangxian uplift zone three upwarps surrounding one downwarp can be observed, namely the Shuangyao, Xiaohanzhuang and Dadongzhuang upwarps, and the Baitangkou downwarp. All these structures constitute the basic feature of the basement of Tianjin area. There exist two sets of fractures, one being compressional --compressional torsional fractures parallel to the NNE uplifts and depressions, and the other tensional--tensional torsional fractures in NNW direction. The former set in-

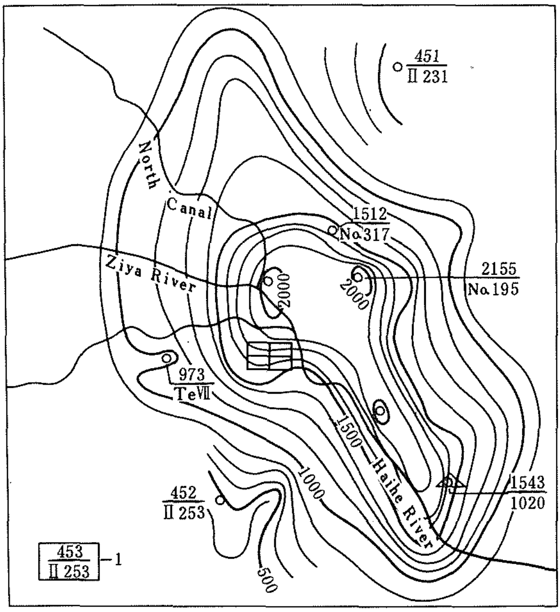

Fig. 1. Contour map of land subsidence in the urban area of Tianjin (1959-1982)
1- subsidence value (mm)/Bench mark

cludes those in Hangu, Dacheng, northe Tianjin, west and east Baitangkou, madong, and east Cangxian; the latter includes the Haihe River fractures. (Fig. 2)

The NeoCathaysian tectonic system was basically formed since the middle-late Jurassic till the late Tertiary, being still active up to the present in local places. The North China Plain began to subside to a great extent in the Cenozoic, receiving loose sediments to a great thickness.

II. Geology and hydrogeology of the Quaternary system

In the studied area, the loose cover in mainly composed of the Quaternary and upper Tertiary sediments, whereas the basement is formed by the Sinian, Cambrian, Ordovician and Carboniferous-Permian rock formations. The rocks at the axial part of the Cangxian uplift are relatively old, while those on either flanks are young. The sharp settlement occurring in the Yanshanian movement has provided an environment for the Cenozoic sedimentation of great thickness, which varies in different structural positions, for instance the axial part of the Cangxian uplift has a thickness ranging from 800 to 1100 metres, whereas the depressional part up to several thousand metres. The Quaternary sediments, composed of clayey soil and sands, are about 600m thick. The top 50 metres are alternating marine and continental sediments of varying lithologic characters and poor engineering geological properties, mostly highly compressional. At the section of 50-200m are medium-low com-

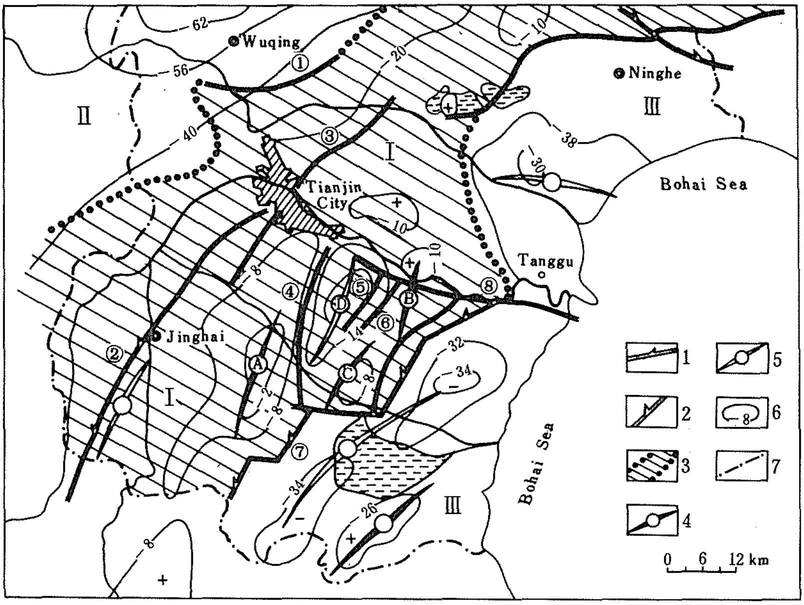

Fig. 2. Basement structure of Tianjin area

I-Cangxian uplift; II-Jizhong depression; III-Huanghua depression; 1,2-fault; 3-boundary between uplift and depression; 4-upwarp; 5-downwarp; 6-Bouguer anomaly isoline; 7-boundary of urban area; A-Shuangyao upwarp; B-Dadongzhuang upwarp; C-Xiaohanzhuang upwarp; D-Baitankou downwarp; (1)-Hangou fracture; (2)-Dacheng fracture; (3)-North Tianjin fracture; (4)-West Baitangkou fracture; (5)-East Baitangkou fracture; (6)-Mafang fracture; (7)-Cangdong fracture; (8)-Haihe River fracture.

pressional sediments mainly of fluvial facies. Below 200m, the sediments are of lacustrine facies, which change slighly in lithologic character and are low compressional.

Within the Quaternary system, one watertable aquifer and five confined aquifers can be identified. The depth of the bottoms of these aquifers are as follows:

Watertable aquifer	5 - 6m
Confined aquifer I	50 - 60m
Confined aquifer II	160 - 170m
Confined aquifer III	250 - 300m
Confined aquifer IV	380 - 410m
Confined aquifer V	500 - 600m

Historical background and present situation

I. Groundwater development

The city of Tianjin saw the development of shallow groundwater early in the 19th century, and that of deep groundwater in 1923. Before 1949 the year of the founding of New China, there were only 51 deep wells, yielding 40,000m^3/day for water supply. After the Liberation of China, particularly after 1958, the extraction of groundwater has been increasing with the ever developing industry and agriculture and the improving living standard of the people in Tianjin. Up to the present, the number of deep wells counts more than a thousand.

The confined aquifer II, being the main source of cooling water for industry in the urban area of Tianjin, provides 50% of the annual pumpage.

The exploitation of groundwater exhibits three peculiarities:

1. Being concentrated in place.

Most of the production water wells are concentrated in the industrial areas along both banks of the Hiahe River. Within an area of less than one kilometre there are 23 production wells, the intensity of the exploitation being as high as $369m^3$/hour.km^2 in the peak season.

2. Being concentrated in specific layers.

As Aquifer II has a shallow depth, great thickness and wide distribution, as well as fine water quality and lower temperature, it has been the main source that the pumping is concentrated upon for water supply. For instance in 1970, the production water wells reached a number of 263, making 52% of the total number of wells in Tianjin. The yielding of these wells occupied 59% of the total extraction.

3. Being concentrated in time.

Take the Dazhigu industrial area as an example, the pumpage in the peak periods is five times that in the dull seasons. Because of the irrational development of groundwater, the water level goes down year after year. The water level in the wells tapping the Aquifer II has generally drawn down 50-60m, occasionally even more, from its original height of plus 1.5m when the wells were completed in 1923. (Fig. 3)

II. Land subsidence

According to the history of groundwater development, the records of water table fluctuation and the rate of settlement, land subsidence in the urban area during the several decades can be divided into four stages as follows:

1. 1923-1957, the starting period of groundwater development and land subsidence.

The information obtained from the past 30 bench marks has shown that the surface subsided to a greater extent round a production well than remote from it. In the former case, the annual cumulative amount was about 7.1-12.3mm, whereas in the latter case, only a few millimetres. Generally speaking, the land subsidence has not brought about any obvious harms.

2. 1958-1966, the intensifying period of groundwater development and the preliminary formation of the settlement center.

The exploitation of groundwater was increasing during this period, forming a cone of depression in Aquifer system II. Simultaneously seven centers of land subsidence with an annually accumulative amount of 30-46mm appeared at Baimiao, Beizhan and several other places,

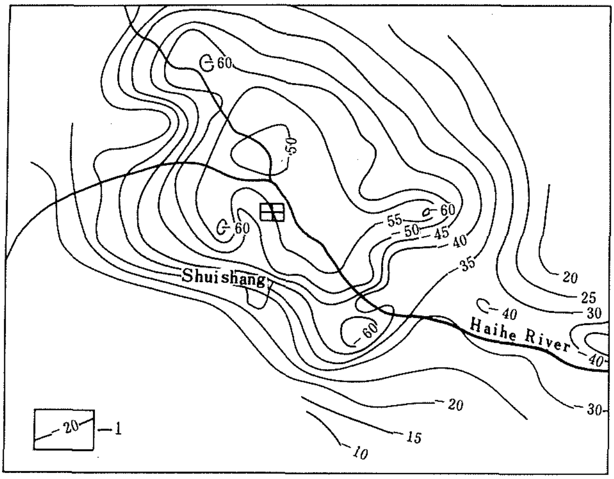

Fig. 3. Cone of depression in Aquifer system II
1-isoline of piezometric level of groundwater (m)

coinciding roughly with the cone of depression. The area of settlement expanded year after year, resulting in the phynomena of cracking in the buildings, breaking of draining pipe lines, and elevating of well tubes.

3. 1967-1975, the period of sharp development of land subsidence.

The settling rate of the ground was accelerated. For example, bench mark No. 195 gave an average annual settling amount of 54mm before 1971, but 163mm during the period from 1971 to 1975. The subsiding area enlarged greatly, having an accumulative settling amount over 600mm and expanding from 1.15km^2 in 1971 to 53km^2 in 1974. The harmful effect was considerably serious. Rain water was collected in the center of the settlement up to one metre deep, and inverse flow occurred in the drains.

4. 1976-1980, the slowing-down period of land subsidence.

The alleviation of land subsidence during this period was due to on the one hand the prohibition of drilling new water wells issued by the municipal government and on the other hand the readjustment of the stress of the earth's crust.

Analysis of the cause of land subsidence

I. Overwithdrawal of groundwater, particularly from the confined aquifer system II, being the main cause of land subsidence in Tianjin.

This can be proved by the occurrence, development and distribution of land subsidence in the urban area.

1. The land subsidence versus time is closely related to the period of heavy exploitation of the confined aquifer system II. For example, as indicated by the bench mark No. 284, the settling amounted to 106mm in a year's time from November of 1977 to the same month of 1978. Of

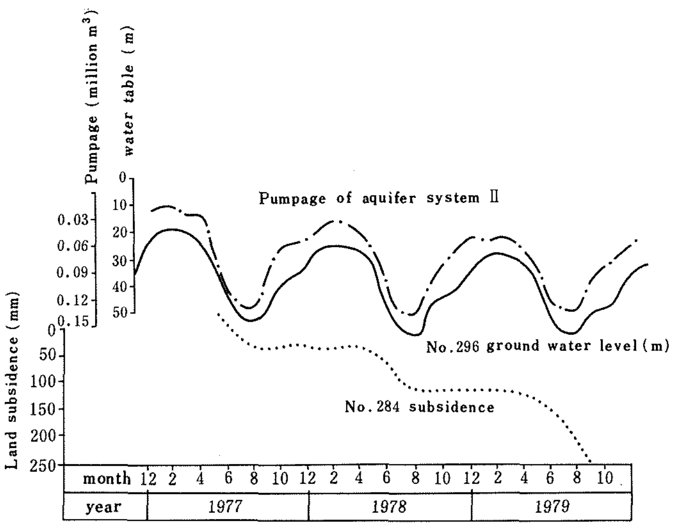

Fig. 4. Correlation of water table of Aquifer System II with pumpage and land subsidence

the total accumulative amount of 106mm, 97mm of settling occurred in the heavy pumping season from May to September, making 93% of the total annual amount of subsidence.

The land subsidence versus time forms a curve with regular depressions and recoveries in a duration of several years. (Fig. 4)

2. Aquifer system II and its influenced zones constitute the main compressional layer causing the land subsidence.

In recent years the information obtained from the bench marks has proved that the deformation amount of the strata within the range of aquifer system II and its influenced zones occupies 50% of the total land subsidence.

3. The center of land subsidence is related to the distribution of aquifer system II.

The confined aquifer system II in the alluvium passes through the urban area in a NW-SE direction. The old channel is composed of alternating sand and clayey soil of similar thickness. Under pumping, the clayey soil layer drains toward the upper and lower sand layers, the pore-water pressure disperses rapidly, resulting in considerable compaction of the clayey soil layer. Five settling centers of the urban area of Tianjin are all located in such zones. In contrast, the sand layers are relatively thin at the corresponding depths in the interfluvial land mass, which constrains the pumping of groundwater, such as in the SW and NE parts of the urban area where the least land subsidence occurred.

II. The effect of strong earthquake on land subsidence

In 1976, an earthquake registered 7.8 on the Richter scale took place in Tangshan, Hebei Province. The epicenter was pinpointed about 100km away from the studied

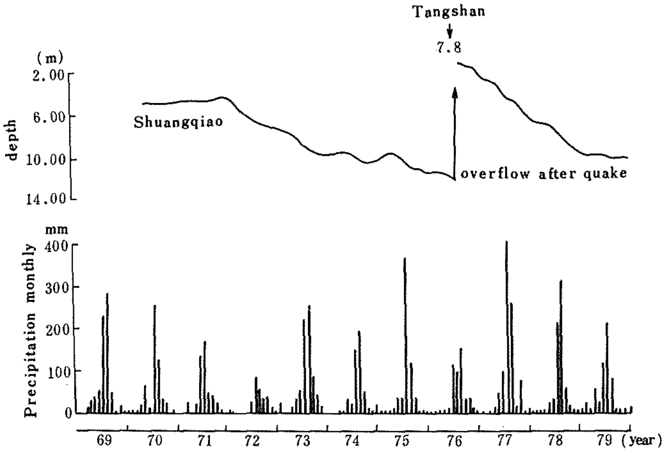

Fig. 5. Monthly average water level in wells tapping saline aquifers in Tianjin

area at the composite position of the NNE NeoCathaysian tectonic system and the W-E structure system.

The authors consider that owing to the abnormal changes of the stress of the regional structures, the strong shock affected the water level fluctuation of the confined aquifers during the whole pregnant period of the earthquake, and was one of the factors not to be neglected.

1. From the observation on the non-exploited saline aquifers by the Tianjin Municipal Bureau of Seismology, one can see the consequent water level variation of the confined aquifer due to the stress anomalies of the earth's crust before and after the strong quake. The observation borehole 64m deep is sited in the Hangu area, Tianjin, 55k m distant from the epicenter. The water level in the hole began to go down in January 1972, 5.44m per year till 1976 when reached its lowest level right before the shock. And after the shock the water level recovered immediately, even gushed out. (Fig. 5)

Though the precipitation of 1975 in Tianjin increased 141.1mm than that of 1974, the water level in the observation well still declined somehow. The fluctuation of water level was not affected by the extraction of groundwater, nor was it controlled by rainfalls. This could only be explained by the abnormal changes of stress before and after the earthquake.

2. Similar to the fact that the lowering down of the water level in a confined aquifer which destroys the stress balance of the soil mass will lead to land subsidence, the variation of water table during the pre- and post-quake periods will bring about the same effect.

Land subsidence can not be explained only by the intensification of groundwater extraction, according to large amounts of data obtained by accurate levelling measurements for several years before and after the earthquake in Tangshan. It is thus considered that:

2.1. During the pregnant period of the strong quake, the stress of the earth became concentrated round the epicenter, where the groundwater was forced to rise up and the land surface was uplifted slightly, as shown in Fig. 6.

Remote from the epicenter area, reversal of stress occurred in the earth's crust, which was under tension condition, the intrastratal pressure reduced, leading to a pressure drop of the pore water. We should say that the fluctuation of the confined aquifers in the area is the result of both groundwater extraction and the stress of the tectonic structures. In fact, the year 1975 before the strong earthquake saw the highest rate of annual subsidence of the ground surface in the urban area of Tianjin. (Fig. 7)

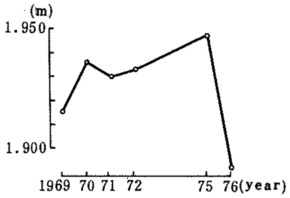

Fig. 6. Elevation Change at Nanbao

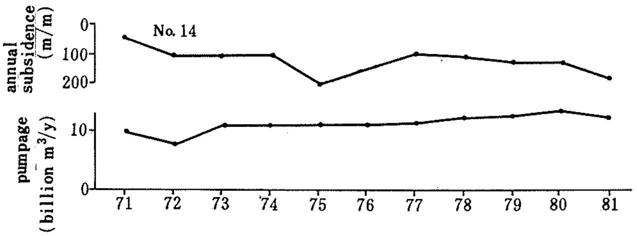

Fig. 7. Correlation between pumpage and land subsidence

2.2. When earthquake took place in Tangshan, the earth's stress released suddenly. The land in the vicinity of the epicenter deformed violently, with the maximum uplifting of 1.3m, whereas in the distant areas from the epicenter the deformation of the land decreased with the attenuation of the stress expanding outwards from the epicenter. For instance, the land surface settled down 1.3m before and after the shock at the levelling point No.35_2, Shanjin; 0.93m at point No.36, Hangu-Shanjin; and 0.3m at the trigonometric levelling point, Chadian.
In contrast, only several millimetres of subsidence, which was within the range of allowable error, was observed at the Zhangdaukou point, 110km from the epicenter.

2.3. Readjusting of the earth's stress took place after the quake. The physical phenomena of elastic rebound of the land occurred, leading obviously to the reducing rate of settling, which was quite in contradiction with the viewpoint that overextraction of groundwater was the only cause for land subsidence.

The pumpage of groundwater in 1980 showed that it surpassed considerably that in 1975, but the settlement shown by the levelling point did not increase with the increasing pumpage. On the contrary the land subsidence slowed down under this circumstance as shown by the measurements in the following table:

Levelling point No.	Settling amount (mm) 1975	1980
284	180	82
195	203	103
314	201	119

In Tanggu area, there was a noticeable change in land subsidence in 1977 after the Tangshan quake: the data from thirty levelling points were then studied, it was found that the data from the seven points among the thirty showed no subsidence at all, but uplift instead. The remaining twenty-three points showed though slight subsidence approximately 150mm less than the annual subsidence before the quake. The average value of subsidence given by the measurements at the thirty points was only 9.6mm. From the curve of subsidence versus time for the levelling points at Sanbaidun, Xinhe Boat Plant and three other places, a staircase-like curve can be seen (Fig. 8). This phenomenon was the result of readjustment of the earth's stress after the Tangshan earthquake.

The effect of the Tangshan earthquake on the studied area is to a certain extent related to the different structural positions of the area. In the duration of several years before and after the shock, the land subsidence induced was about from some tens to one hundred millimetres.

It is thus considered that a strong earthquake in its active period is a nonnegligible factor for land subsidence within the range of its influence.

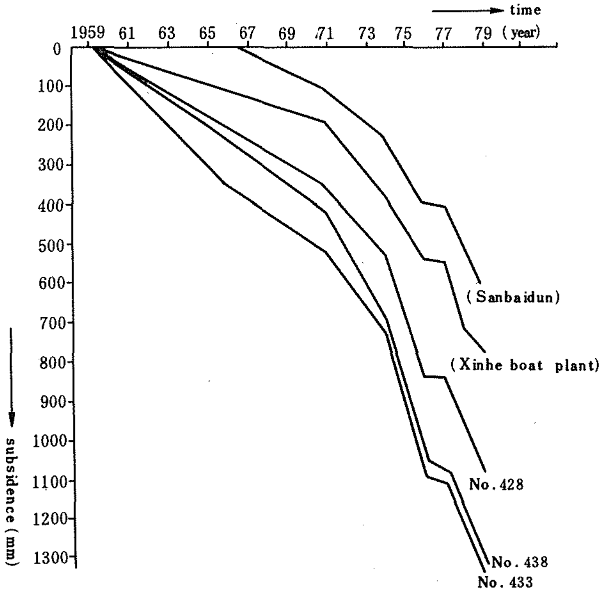

Fig. 8. Curve showing the subsidence at some of the bench marks in Tanggu District

III. The influence of the compaction of less consolidated soil beds on land subsidence

Within the depth ranging from 5 to 15m in the studied area are widely distributed grey--dark grey clayey soil and muddy clayey soil, which is the first layer of marine facies. Its water content is about 30-42%, porosity >1.0, liquidity index mostly >1.0, and compressibility coefficient approximately 0.05cm^2/kg. It is a kind of moderately compacted soil in a state of soft and plastic flow, being medium-high compressional. The consolidation pressure test on the soil for the previous stage indicates that the soil bed is less consolidated, in which still exists residual porwater pressure. Under its selfweight, the soil bed trends to become normally consolidated, causing land subsidence.

Conclusion

Land subsidence of Tianjin can be attributed to two factors: man's activity such as overextraction of groundwater and crustal movement such as earthquake. In Tianjin, the former plays the main role and the latter appears to be a nonnegligible factor, since strong earthquakes display a specific form of crustal movement, resulting in apparent settlement in different stages. Therefore , in working out the control measures , identifying the inducing factors of land subsidence is of practical importance.

PROLONGED SUBSIDENCE OBSERVATIONS IN SOFT SEDIMENTS UNDER A SAND LOAD

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Abstract

In the recently reclaimed polder Flevoland for town building purposes a hydraulic fill of 1 m of sand is supplied on top of the soft to very soft holocene sediments (thickness about 4 to 6 m). This fill brings about a long lasting overpressure in the pore water, due to the low hydraulic conductivity of the holocene sediments. Subsidence occurs as a result of the load by the sand fill and the decrease of the piezometric head in the underlying pleistocene sand. At a number of sites the decrease of the pore water pressure and the subsidence in the course of time are recorded. For two sites, having the longest lasting period of records (11 years) a comparison is made between the recorded values and the results of a calculation by a simple simulation model.

Introduction

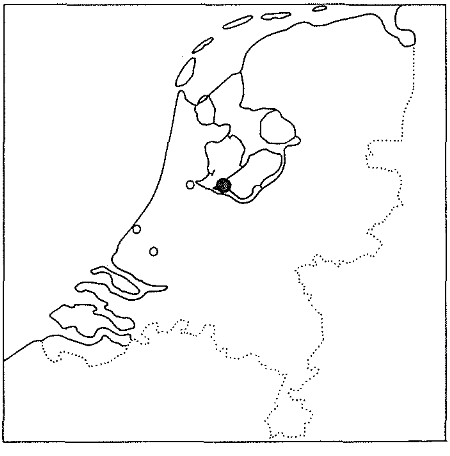


Fig. 1. Location of the research area

In the centre of the Netherlands new polders are reclaimed from the IJsselmeer (Fig. 1). A part of the last reclaimed polder is used for the construction of new towns. They are built on soft to very soft holocene clay and peat layers (4 to 6 m), underlain by a thick layer of pleistocene sands (200 to 300 m).

A hydraulic fill of about 1 m of sand is installed on top of these soft layers to have good drainage conditions and a well passable surface in wet periods. The load by this fill, the lowering of the groundwater table and the drop in piezometric head in the course of time cause compaction and subsidence of the soft layer. Construction purposes require a prediction of the subsidence and its course in time as accurate as possible to be able to design adequate constructions.The sand fill brings about also pore water overpressure in the soft layer. This overpressure decreases very gradually only, due to the very low hydraulic conductivity of these layers.

At nine sites the subsidence and the pore water pressure are observed. The records of two sites, observed for 11 years already, are compared with the results of the calculations by a simple simulation model.

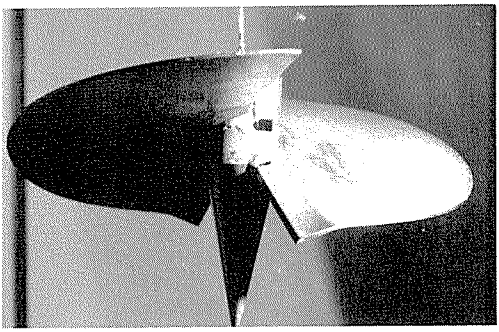


Fig. 2. The type of plate used in the subsidence measurements

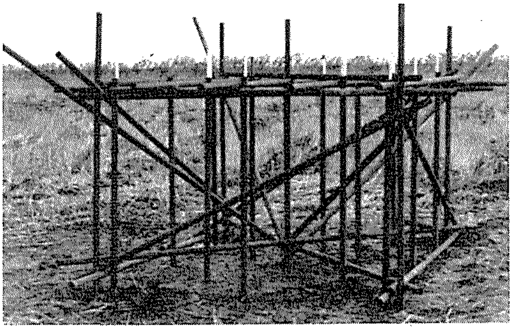


Fig. 3. A measuring site before the supply of the sand fill

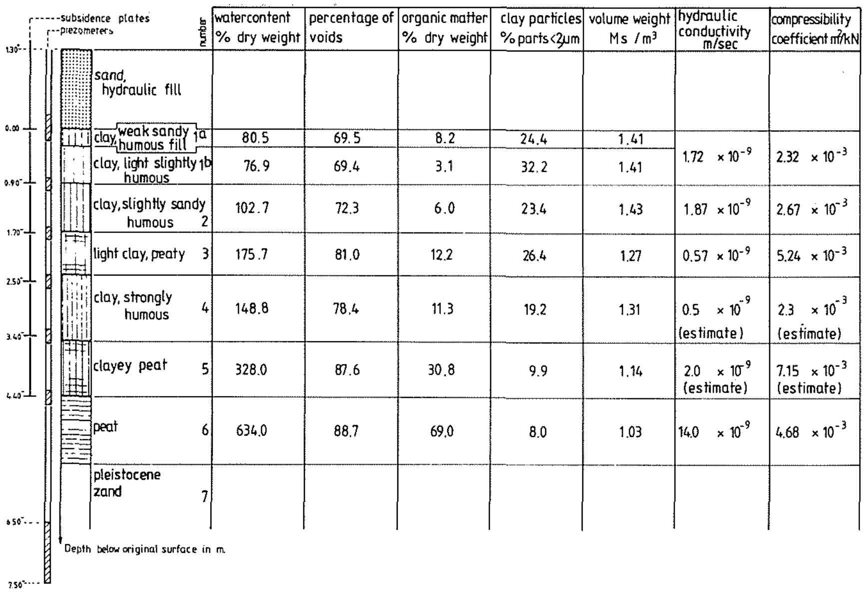
Lay-out of the measuring sites

The holocene consists of various deposits, varying both in geological origin and in density and texture. This gives rise to differences in compaction. Besides, the influence of a load on the compaction decreases with depth in an extended area, as the initial grain stress increases with depth and the effect of the load is equal throughout the profile, so the ratio between the initial grain stress and the grain stress under the load diminishes with depth. Hence, to arrive at a good prediction of the subsidence, it is necessary to observe the compaction of the distinct layers.

The total subsidence, being the subsidence of the original surface below the hydraulic fill, is observed by an iron plate (1.0 x 1.0 m). The compaction of the distinct geological deposits is observed by screw plates (Ø 0.25 m), brought down into the soil to the boundaries between these layers (Fig. 2). An iron rod (Ø 8 mm) is attached to the screw plates and the surface plate. They extend above the surface of the sand fill and are able to move inside a plastic mantle pipe (Ø 16 mm) to avoid skin friction by the compacting layers.

The pore water pressures are recorded by open piezometers, made of plastic pipe (Ø 19 mm) with a perforation over the bottommost 0.20 m. The perforation is envelopped by a geotextile and gravel. Just above the perforation the bore hole is sealed with bentonite. Calibration of the open piezometers with electric piezometers showed a good agreement for the long term observations. The piezometer in the pleistocene sand consists of an iron pipe (Ø 25 mm) and a copper filter over the bottommost metre. The top of this filter is used as a bench mark, as it does not subside.

All the iron rods and the piezometers are protected by iron mantle pipes (Ø 50 mm) to avoid damage during the hydraulic filling and possible other disturbances afterwards. These pipes are connected to each other by iron pipes in various directions. This framework got the nick-name "hedge-hog-pit" ever since (Fig. 3). This protection appeared to be efficient. A test without such a protection was not proof against the forces during the hydraulic fill.



	number	watercontent % dry weight	percentage of voids	organic matter % dry weight	clay particles % parts<2µm	volume weight Ms / m³	hydraulic conductivity m/sec	compressibility coefficient m²/kN
sand, hydraulic fill								
clay, weak sandy humous fill	1a	80.5	69.5	8.2	24.4	1.41	1.72×10^{-9}	2.32×10^{-3}
clay, light slightly humous	1b	76.9	69.4	3.1	32.2	1.41		
clay, slightly sandy humous	2	102.7	72.3	6.0	23.4	1.43	1.87×10^{-9}	2.67×10^{-3}
light clay, peaty	3	175.7	81.0	12.2	26.4	1.27	0.57×10^{-9}	5.24×10^{-3}
clay, strongly humous	4	148.8	78.4	11.3	19.2	1.31	0.5×10^{-9} (estimate)	2.3×10^{-3} (estimate)
clayey peat	5	328.0	87.6	30.8	9.9	1.14	2.0×10^{-9} (estimate)	7.15×10^{-3} (estimate)
peat	6	634.0	88.7	69.0	8.0	1.03	14.0×10^{-9}	4.68×10^{-3}
pleistocene zand	7							

Fig. 4. Data on soil parameters and the depth of the plates and the piezometers at site I

Soil parameters

Information on the soil profile, the situation of the plates and the piezometers and the soil parameters at site I is given in Fig. 4. The soil parameters were determined in cores from sample borings. The water content in this water saturated soil is high, as is the percentage of voids. The

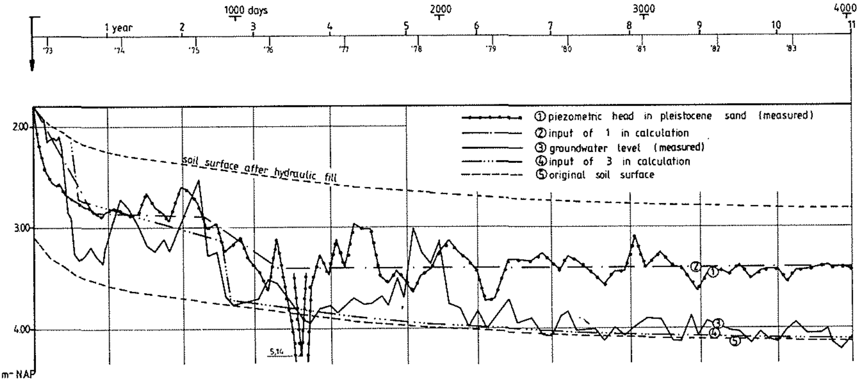


Fig. 5. The course of the groundwater level and of the piezometric head in the sandy subsoil at site I

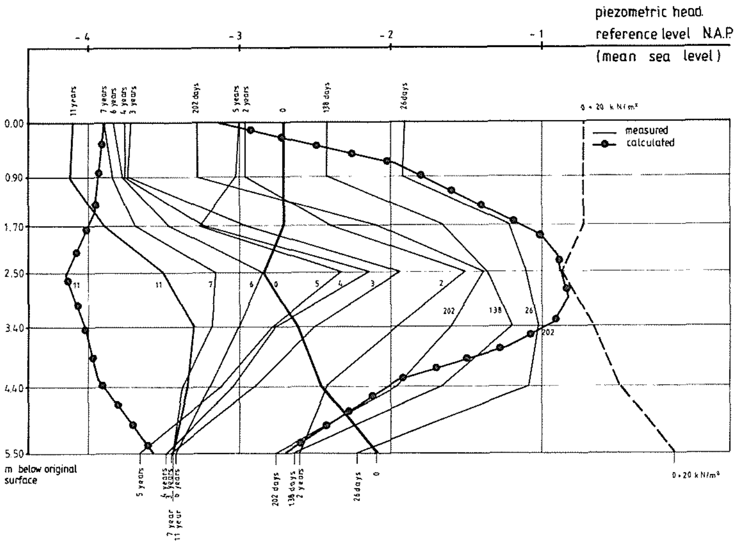


Fig. 6. The calculated water overpressure at the moment of loading, the same pressure at a number of days after loading and the calculated water overpressures at two points in time

percentage of organic matter increases with increasing depth, while the volume weight decreases. In the clayey peat a lot of organic matter with a disturbed fabric is present, eroded elsewhere. The hydraulic conductivity and the compressibility are determined in a conductivity and an oedometer test at various overburdens. The values in Fig. 4 are determined at an overburden of 20 kN.m^{-2}. No tests were done in layer 4 and layer 5. Therefore, the hydraulic conductivity and the compressibility were estimated from neighbouring borings. Comparable soil parameters were found at site II.

Pore water pressure

Next to the load by the sand fill the groundwater table and the piezometric head in the pleistocene sand are decisive also for the water pressures in the holocene layers. The variations in this level and in this head in the course of time are shown in Fig. 5. The sand surface subsides and the groundwater table drops also on average at the same rate. The piezometric head in the pleistocene sand drops in the course of time as a result of the progressing reclamation of the polder.

The levels in the piezometers in the holocene layers at site I at a number of days are given in Fig. 6. The initial course of these levels throughout the profile (before filling) is indicated by day 0. In a profile without pore water overpressure or underpressure everywhere in the profile the same level will be present. At site I the piezometric head in the pleistocene sand exceeds the level of the groundwater table, and, hence, the levels in the piezometers increase with increasing depth. The volume weight of the hydraulic fill is 20 kN.m^{-2} before a subsurface drainage system was installed, as the sand was saturated with water. After that the volume weight is 16 kN.m^{-2}. The theoretical water overpressures just after the fill is completed are indicated by 0 + 20 kN.m^{-2} in Fig. 6.

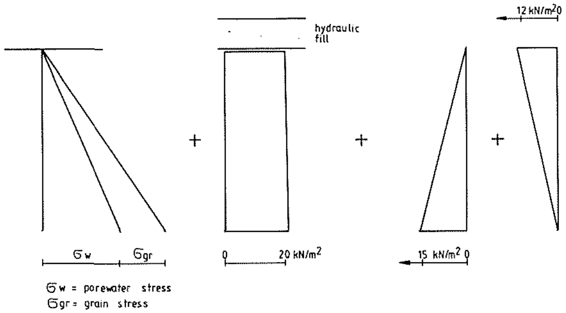


Fig. 7. Increase and decrease of the pore water pressure applied in the simulation model

The pore water over- or underpressure is calculated by the consolidation formula of Terzaghi (Terzaghi, 1951), reading

$$u = \sum_{m=0}^{m=\infty} \frac{4p}{(2m+1)\pi} \sin\left\{\frac{2m+1)\pi}{h} z\right\} e^{\frac{-k}{m_v j \gamma_w} \left\{\frac{(2m+1)\pi}{h}\right\}^2 t} \qquad (1)$$

in which
u = pore water overpressure $(kN.m^{-2})$
p = load or initial pore water overpressure $(kN.m^{-2})$
h = thickness of the layer (m)
z = depth between zero and h (m)
k = hydraulic conductivity $(m.s^{-1})$
m_v = compressibility coefficient $(m^2.kN^{-1})$
γ_w = volume weight of water $(kN.m^{-3})$
t = time (s)

Based on this formula a simple simulation model was developed. The thickness of the various geological layers in the profile was converted in such a way, that

$$\frac{k}{m_v \gamma_w} \left\{\frac{(2m+1)\pi}{h^2}\right\} t \qquad (2)$$

is equal in each layer (Schmidt, 1924; Langejan, 1962). The converted thickness becomes

$$h_2 = h_1 \frac{k_2 m_{v1}}{k_1 m_{v2}} \qquad (3)$$

The pore water overpressures are calculated in three phases (Fig. 7):
- an overpressure of 20 $kN.m^{-2}$ is applied, resulting from the load by the hydraulic fill, being supposed to be supplied in 0.15 year,
- a decrease of the piezometric head in the pleistocene sand by 15 $kN.m^{-2}$, applied in two steps as indicated in Fig. 5 and
- the groundwater table is lowered by 12 $kN.m^{-2}$ as a result of the subsidence of the surface, applied also in two steps as indicated in Fig. 5.

The results of the calculations at a number of days are given in Fig. 8 as the difference between the calculated and the measured piezometric heads. At two points of time the course of the calculated head throughout

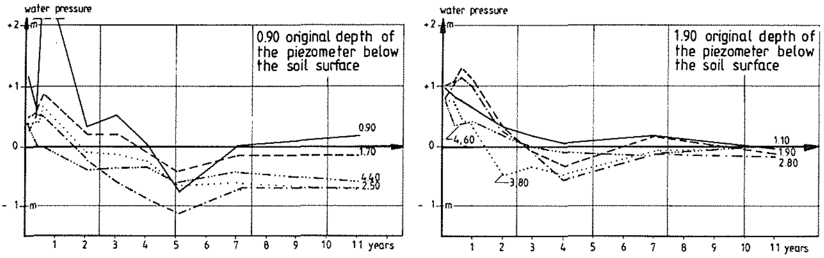


Fig. 8. The difference between the calculated and the measured pore water pressure; left site I, right site II (calculated minus measured)

the profile is given in Fig. 6. To both figures the following remarks can be made:

- the measured piezometric heads at a depth of 0 and 0,90 m are equal (Fig. 6). The physical ripening of these soft sediments results among other things in crack formation, bringing about a relatively good permeability. This could not be taken into account in the calculations;
- the raise of the pore water pressures in the early phase of the consolidation stays behind, as the adjustment of the open water levels in the piezometers to the actual piezometric head takes too much time, due to the low hydraulic conductivity. This period of adjustment takes some two to four months, in general. For good results in this early phase electric piezometers have to be preferred;
- the calculated pore water pressures are too high as compared with the measured values for the first two to four years. After that, the calculated values are below the measured values. After about seven years the differences between the calculated and the measured values become rather constant.

Site II is analysed in the same way. The differences between the calculated and the measured values are also given in Fig. 8. The soil profile and the soil parameters are well comparable with those at site I. At this site the agreement between both values is much better, though the calculated piezometric heads are also higher than the measured ones in the beginning. After seven to eight years there is a good agreement between both values.

The most reasonable explanation for the differences between the calculated and the measured piezometric heads and its variations is that the values for the hydraulic conductivity and the compressibility change during the consolidation process. The value of both parameters will decrease in time.

Subsidence

In the simulation model the decrease of the pore water pressure is considered as an increase in grain stress for the calculation of the subsidence. It is calculated by the formula of Terzaghi/Keverling Buisman (van der Veen et al., 1981), reading

$$S = T\,\{(\frac{1}{c_p} + \frac{1}{c_s}\log\,t)\ln\frac{P_g}{P_1} + (\frac{1}{c'_p} + \frac{1}{c'_s}\log\,t)\ln\frac{P_2}{P_g}\} \qquad (4)$$

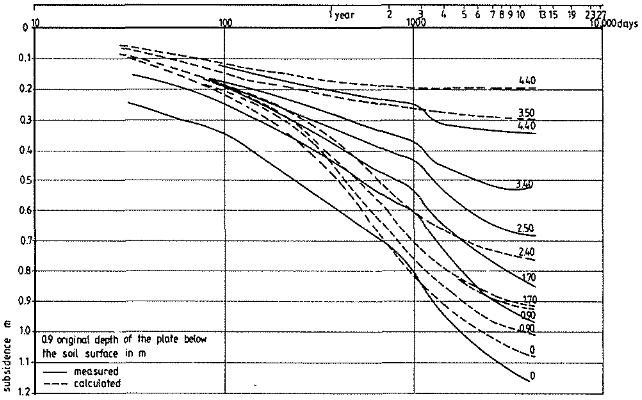


Fig. 9. The calculated and the measured subsidence at various depths at site I

in which

S = subsidence (m)

T = initial thickness (m)

t = time since loading (days)

P_1 = initial grain stress ($kN.m^{-2}$)

P_g = preconsolidation grain stress ($kN.m^{-2}$)

P_2 = final grain stress ($kN.m^{-2}$)

$\frac{1}{c_p}, \frac{1}{c_s}, \frac{1}{c'_p}, \frac{1}{c'_s}$ = compressibility coefficients (-)

$P_2 = P_1 + \Delta P$, in which ΔP is the load by the sand fill ($kN.m^{-2}$).

The calculation is done in two phases. The first phase is the calculation of the subsidence caused by the load of the hydraulic fill. The decrease of the pore water overpressure, due to this fill is considered as an increase in grain stress. The second phase is the calculation of the subsidence, due to the decrease in the piezometric head in the pleistocene sand. This causes also an increase in the grain stress and, hence, subsidence.

In the first phase the stress level in the soil increases, therefore in the second phase the initial stress P_1 in form. 4 is raised by 20 $kN.m^{-2}$ (the load of the sand fill). The lowering of the groundwater table causes no increase in grain stress, because the soil surface subsides at the same rate (Fig. 5).

The results of the subsidence calculations together with the measured values are given in Fig. 9 for site I. The following conclusions and remarks can be made:

- The calculated subsidence rate is too low at first. This is similar to the differences between the calculated and measured values of the piezometric heads. Just after the supply of the hydraulic fill the pore water overpressure is equal to the load. This means that the soil reacts as an uncompressible layer. For loads on an infinite area theoretically no subsidence will take place;
- a second reason for an initially lower subsidence rate could be that the hydraulic fill is heavier than assumed;
- the compressibility coefficients in form. 4 are measured within the range

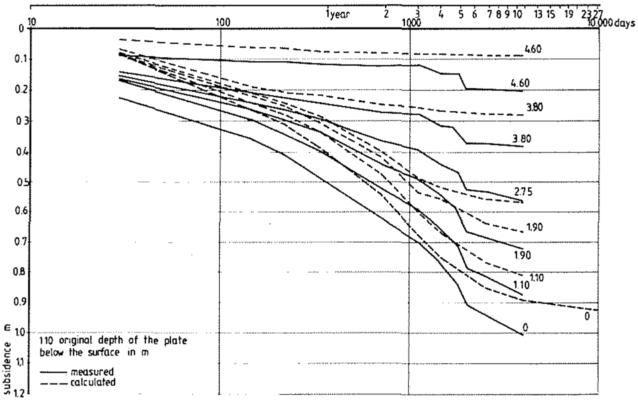


Fig. 10. The calculated and the measured subsidence at various depths at site II

of the expected increase in grain stress. They are kept constant in the calculation, though they will be dependant of the increase in grain stress;

- the calculated total subsidence does not exceed the measured subsidence. If the pore water overpressure, still present in the soil, is taken into account, the measured values will even more exceed the calculated values. The weight of the overburden of 20 kN.m^{-2} seems to be too low;
- the sudden changes of the rate of the measured values are caused by drainage, extremely dry periods or a temporary extreme lowering of the piezometric head in the pleistocene sand (Fig. 5);
- the prediction of the subsidence of the various layers is not correct. In the clayey peat layer (3.50-4.40 m) it is too small. The compressibility coefficients of this layer differ from those of locations nearby. In the strongly humose clay layer (2.40-3.50 m) it is too large. For the calculation the layer 2.40-3.50 m is taken. For comparison a reduction of 0.90/1.10 of the calculated value should be taken. Even with this reduction the calculated subsidence is too large;
- the calculated subsidence of the topmost layer is smaller than the measured value.

The results of the subsidence calculations together with measured subsidence at site II are given in Fig. 10. Also in this figure it appears that the calculated subsidence differs in the same way from the measured one as at site I.

Conclusion

The most important conclusion is that even with a simple simulation model a first broad approximation of the subsidence can be made. However, the accuracy of the prediction is highly dependant on reliable values for the hydraulic conductivity, the compressibility coefficients and the volume weight of the load. Next to this, possible changes in the level of the groundwater table and the piezometric head in the pleistocene sand below the compressible layers have to be known well.

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LAND SUBSIDENCE IN THE NETHERLANDS

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Abstract

Part of the Netherlands is situated below Mean Sea Level. Therefore subsidence of dikes, etc, protecting the country against the water, may be dangerous. Several repeated levellings have been carried out to study land subsidence. The vertical crustal movements, resulting from the first-order levellings (1875-1885, 1926-1940, 1950-1959 and 1965-1980) are discussed.

Problems related to the maintenance of a network of benchmarks are indicated, one of the problems being the fact that there are no stable rocks in the Netherlands for the foundation of the benchmarks.

A short description is given of the method of hydrostatic levelling, used in the coastal area to transfer Amsterdam Ordnance Datum to natural and artificial islands (off-shore structures) with a very high accuracy.

Since 1962 a subcommission of the Netherlands Geodetic Commission studies natural as well as human caused land subsidence. In this subcommission various sciences are represented. A short survey is given of the problems studied by the subcommission, including the movements caused by the extraction of coal, gas, salt, oil and water.

Introduction

The Netherlands is a small country in north-western Europe, in the Rhine estuary, situated at the North Sea. About 20 per cent of the surface of the Netherlands lies below mean sea level. Even nearly 38 per cent of the country actually lies below high water level. The country is protected against the sea by natural dunes and islands and by man made dikes.

The Dutch people being familiar with the water feel pretty safe behind their dunes and dikes. They have built towns, ports and major industries in the vicinity of the sea and at the moment about half of the total population is living in the low-lying part of the country. The situation proved to be vulnerable: during a dreadful storm tide on 1 st February, 1953 a number of dikes collapsed and large areas in the south-western part of the Netherlands were flooded. Nearly 1800 people lost their lives.

The reconstruction of the dikes started soon, also the Delta Plan was designed. This great hydraulic project includes shortening of the shoreline of the Netherlands by connecting islands in the delta by dams. The existing dikes were raised to a design height taking into account the estimated subsidence of the shore relative to mean sea level.

N.A.P.

At present in the Netherlands all height data are related to N.A.P. (Normaal AmsterdamsPeil = Ordnance Datum of the Netherlands). The history of N.A.P. has been described by Van der Weele (1971).

As early as 1565 in old documents regarding Amsterdam a water level is mentioned that might be the origin of Amsterdam Datum. A hundred years afterwards 8 marble benchmark stones were placed in newly built sluices provided with a groove and bearing the inscription: "Sea-bank's height, being 9 feet 5 inches above town datum".

Amsterdam Datum is expected to have been the height of mean summer hight tide of the IJ, at that time the tidal harbour of Amsterdam being in open connection with the North Sea, via the Zuiderzee. See Fig. 1. A municipal water office was established to measure and register waterheights once or twice per hour. Consequently a long series of tide gauge observations exist, from 1682 to 1930. The registration ended in 1930, because in that year the Afsluitdijk is in construction transforming the Zuiderzee into a lake insensible to tidal effects.

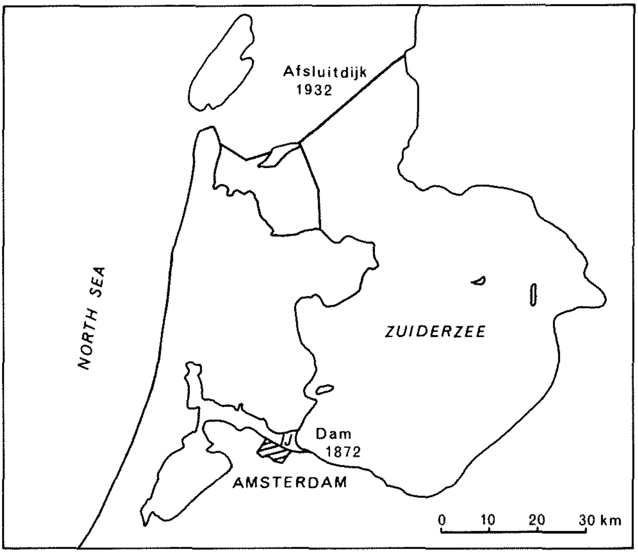

Fig. 1 Amsterdam, having a tidal harbour until 1872.

In the eighteenth and nineteenth century Amsterdam Datum gradually came in use in a larger area, even in a part of the neighbouring countries Belgium and Germany. In 1818 Amsterdam Datum is prescribed as the official Ordnance Datum of the Netherlands to be used as a reference for tide gauge observations and for the determination of the heights of dikes along the rivers. It has been used to define the datum of several levellings ever since carried out over the whole country.

Unfortunately the N.A.P.-stones disappeared, the last of them in 1953. In stead of them a 23 m long foundation-pile has been placed at the Dam in Amsterdam in the centre of the city now acting as fundamental benchmark. Also a second pile at Amsterdam is installed to visualize N.A.P. This pile has been used as a test-pile during the construction of a tunnel. During a test this pile has been loaded with a weight of 850 tons showing no vertical movement.

Relative sea level rise

Along the shore of the Netherlands part of the North Sea 9 tide gauge stations have been observed since 1870, providing us with a lot of valuable information. It was decided in 1829 to maintain the zero points of all tide gauges in the Netherlands at N.A.P.-level. As a consequence the various

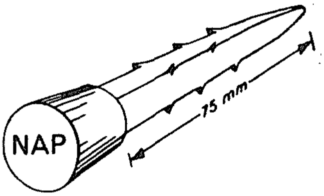

Fig. 2 N.A.P.-benchmark

tide gauge observations do not indicate local (tectonic) movements, but will give the subsidence of the Ordnance Datum at Amsterdam relative to sea level.

The registered observations have been thoroughly studied by a number of scientists. Van Veen (1945) constructed a curve of the rise of half tide level with respect to the N.A.P., based on the observations by the municipal water office at Amsterdam as well as on observations of the tide gauges in the Zuiderzee and along the shore. He computes a rise of sea level of about 17 cm/century.

Thus the assumption of a future relative rise of sea level of about 20 cm/century as estimated by Van Veen and Waalewijn (1961) seems to be safe. This value has played a role in the fixing of Delta heights of the dikes along the shore. However, a very recent statistical analysis of the data of the North Sea tide gauges made by Van Malde (1983) seems to indicate a rise of mean low tide of 10 cm/century and a rise of mean high tide of 25 cm/century. Although a certain margin of safety has been calculated in fixing Delta heights, the question of the rise of sea level asks for careful study.

Subsidence of the land

Another question is whether the other parts of the country rise or subside. This question can be answered on the basis of a comparison of the results of repeated precise levellings of a wide network of very stable founded benchmarks. In the past centuries a large numbers of benchmarks has been placed, but unfortunately all of them have disappeared, even the 8 fundamental benchmarks in the sluices at Amsterdam. The oldest still existing benchmark was placed in about 1880.

At present the Dutch network comprises about 45.000 benchmarks, most of them being round bolts placed in houses and stable buildings like churches, sluices, viaducts, etc. See fig. 2. There stability depends on the foundation of these buildings and on geological conditions, and last but not least on human activities like mining of coal, oil, gas or salt, water withdrawel, reclamation of land, etc. These benchmarks are mainly used for many technical purposes: deriving heights for the design and the construction of roads, bridges, buildings, etc. For these technical purposes relative accuracy of provided heights is more important than absolute accuracy.

However for scientific reasons a number of very stable founded benchmarks with very accurate heights is needed to be able to determine tectonic movements. For that purpose since 1926 a number of specially constructed underground benchmarks has been placed at locations selected on the advice of the Geological Survey of the Netherlands.

The problem is that there is no stable rock on the surface of the country; approxmimately half of the surface of the Netherlands is covered by Holocene sediments of clay, sand and peat, which attain thicknesses up to 20 m. In these areas compaction can be relatively great in relation to the tectonic movements. Therefore the underground benchmarks were founded in the Pleis-

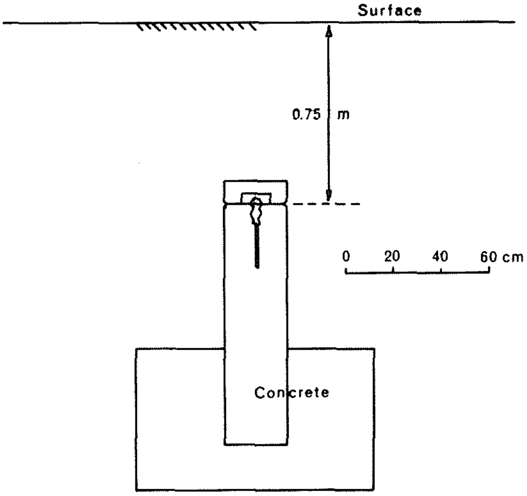

Fig. 3 Underground benchmark

tocene. In places where the Pleistocene layers are found on the surface concrete blocks were constructed (See Fig. 3). In the Holocene regions concrete foundation-piles were placed resting on the sandy Pleistocene underground. Depending on the depth of the Pleistocene the length of the piles may go up to 43 m. At present there are 150 underground benchmarks, of which 100 have been placed since 1950.

The Survey Department of Rijkswaterstaat of the Ministry of Transport and Public Works is in charge of the maintenance of the network of benchmarks, including the publishing of heights. To be able to check the once published heights and to correct them if necessary, the benchmarks are relevelled at more or less regular time intervals. For that purpose 4 first-order levellings have been carried out during the last century, followed by lower-order densification levellings. These first-order levellings took place in the period 1875-1885, 1926-1939, 1950-1959 an 1965-1980 respectively. The adjustment of these levellingnets result in heights with regard to N.A.P. as visualized at Amsterdam; consequently the calculated rise or subsidence of parts of the country is always relative to Amsterdam.

A problem is that the lifetime of most benchmarks is limited. It is found that every year about 2-5 per cent of the total amount of benchmarks disappears. As a consequence the number of benchmarks common to several first-order levellings is rather small: the first and second levelling had approx. 460 common points, there were about 300 benchmarks common to the first, second and third levelling, and all four levellings had only 72 common points.

Edelman (1954) compared the heights of the 460 common points in the first and second levelling mentioned before. He found considerable and irregular subsidences in the northern part of the country, more than 10 cm, and a rise of about 40 mm in 50 years in the southern part. See Fig. 4. He concluded a tilting of the Netherlands along an axis through Amsterdam in WSW-ENE direction.

However, due to instrumentation, the structure of the net, etc. the first

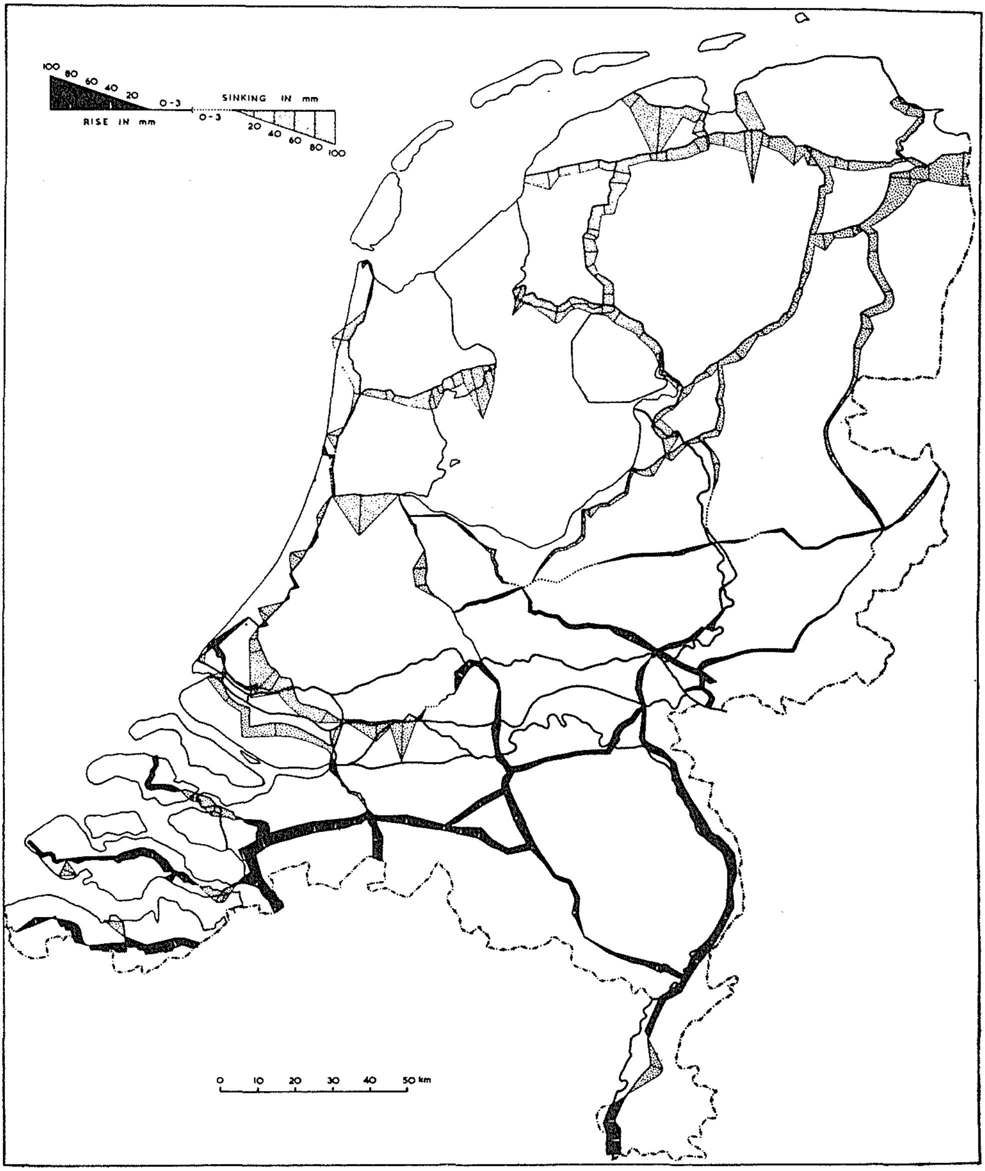

Fig. 4 Differences between the levellings of 1875-1887 and 1926-1940, relative to Amsterdam. The differences are plotted graphically along the lines of levelling. (After Edelman)

levelling and its results are assumed now to be less reliable than the more recent levellings. It should further be borne in mind that the underground benchmarks have been placed since 1926, so the local subsidences may be partly explained by the poor foundation of buildings where the benchmarks were

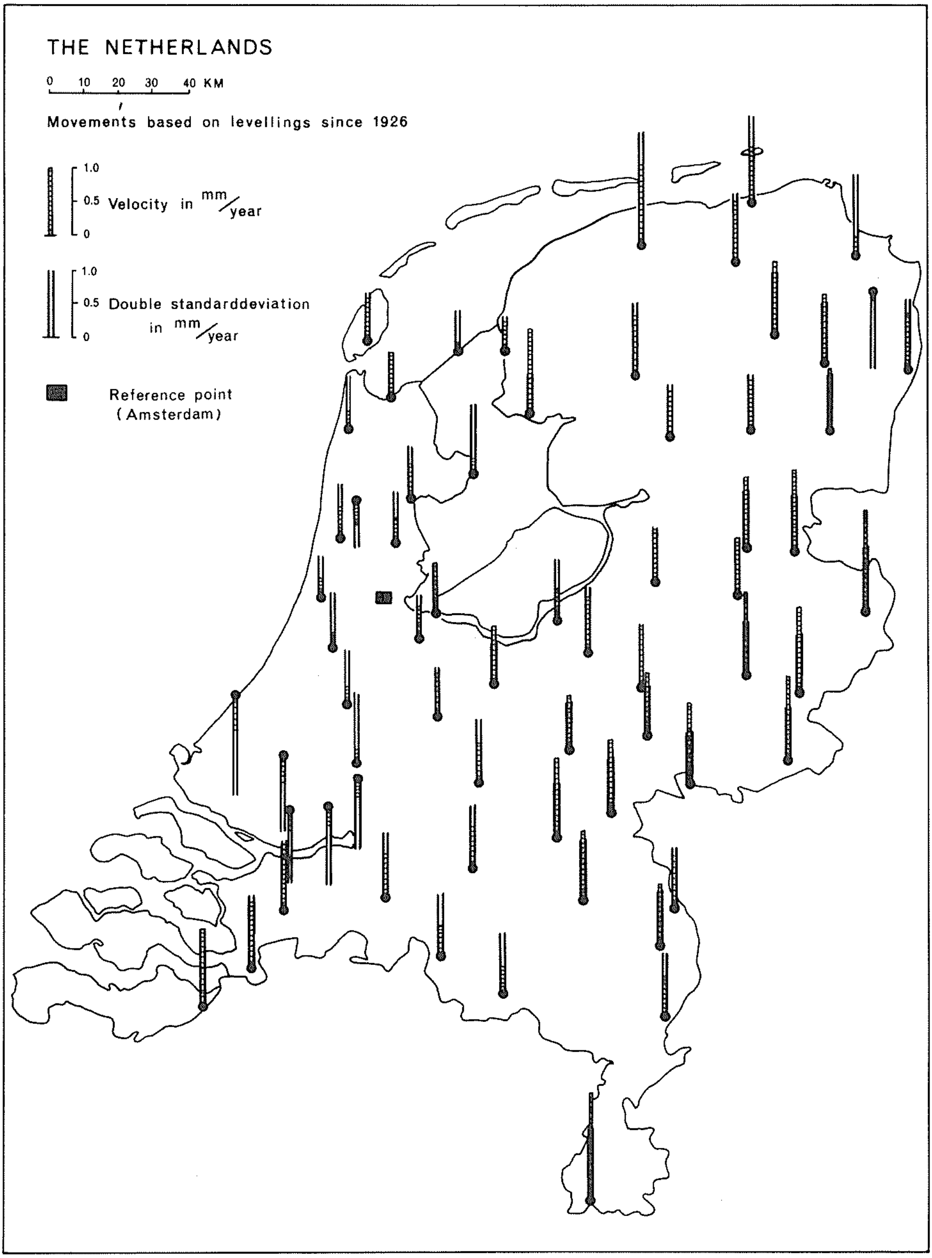

Fig. 5 Preliminary results of comparison of the levellings of 1926-1940, 1950-1959 and 1965-1980, indicating tectonic movements relative to Amsterdam.

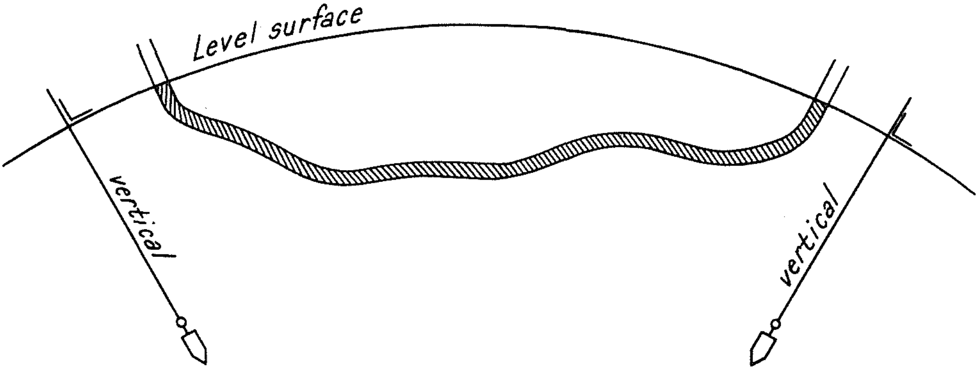

Fig. 6 Principle of hydrostatic levelling.

placed. As a consequence of all of this the results of the third and fourth levelling will not be compared to the first but to the second levelling.

The results of the third levelling indicate no significant change in the respective heights of the underground benchmarks. While adjusting the measurements the levelling net is therefore not fixed on the fundamental benchmark at Amsterdam alone but on all underground benchmarks in the country, their heights assumed to be unchanged. The same method has been applied in the adjustment of the fourth levelling although several underground benchmarks seem to have significantly changed in height. By this method the nets have been transformed being the reason that the resulting heights of the normal N.A.P.-benchmarks are "heights for practical use" and are not suitable for the purpose of scientific research.

At the moment a thorough study of the last 3 levellings is being carried out using a computer. Differences of heights of common very stable points in the 3 levellings, in combination with their precision, will be statistically tested to be able to reach conclusions on land rise or subsidence since 1926. Fig. 5 shows a preliminary result of a comparison of the heights of 66 underground benchmarks included at least twice in the second, third or fourth levelling. The resulting rise or subsidence is expressed as a velocity in mm/year. To roughly indicate the precision of the calculated movements the double standard deviation is also rendered.

Absolute movements

An ever intriguing question is the following: Is Amsterdam stable or not, or in other words: is it possible to compute absolute movements? The answer is: not at present, may be in the future by observing gravity in combination with first-order levellings.

Another possibility to get an idea now of absolute movements is to combine the Netherlands levelling network with networks of neighbouring countries which one supposed to be stable and to adjust them as a whole. It has been decided to try it in that way.

The DGK-Arbeitskreis für Rezente Höhenänderungen (1979) published a Map of Height Changes in the Federal Republic of Germany. It came out that the Arbeitskreis is willing to cooperate with other countries in order to extend this map to Western Europe. The Geodetic Institute of the University of Hannover will act as a central computing centre. After final check of the relevant Dutch levelling data they will be made available to the computing centre.

Hydrostatic levelling

Levelling is normally carried out with the help of an optical levelling instrument and rods. Typical for optical levelling is that a section to be levelled is divided into a number of stretches of 80-100 metres each. The mean square error of first-order levelling is 0.7 mm/$\sqrt{km}$ or better. The usual methods are not suitable to transfer levels to islands or off-shore structures.

The Danish professor Nørlund put the method of hydrostatic levelling for the first time into practice in 1938 and used it with great succes for a levelling across the Great Belt. Afterwards Waalewijn (1964) developed this method and brought it into practice for a large number of off-shore measurements and measurements through rivers and canals.

The method of hydrostatic levelling is based on the law of communicating vessels. Fig. 6 shows the principle. If a pipe is filled with a liquid the surfaces of the liquid in both ends of the pipe will always coincide with a level surface, i.e. the surfaces of the liquid have the same height. There are a number of factors effecting the accuracy. The liquid has to have a constant temperature; if the air pressure is not equal on both sides of the pipe the surfaces of the liquid have not the same height; there is an astronomical effect caused by sun and moon. It is a must that there are no air bubbles in the pipe.

In practice a lead pipe is used with an internal diameter of 10 mm, with walls 3.5 mm thick and with steel wire reinforcements to raise the tensible strength of the pipe. The weight of the pipe having an external diameter of 35 mm is about 3 kg/m. The pipe is always kept filled with water; the level of which can be observed by gauge glasses at both ends of the pipe during measurements.

The Survey Department owns a number of pipes with different lenghts. So levellings can be made with coupled pipes with a total lenght of about 10-20 km. Undoubtedly this device is the heaviest levelling instrument in the world! The pipe is installed in our cable-laying vessel "Niveau", niveau being Netherlands for "level", and laid out and taken in by means of a winch. The accuracy of hydrostatic levelling is high: 0.5 mm per 5 km. This high accuracy is only reached when systematic effects mentioned before are eliminated.

Problems studied at present

It will be clear that it is of vital interest that sea level movements and land subsidence are studied carefully. The Netherlands should have an open eye to future movements to be able to take precautionary measures such as raising the dikes along the sea and the rivers. For that reason the Netherlands Geodetic Commission set up a Subcommission on Crustal Movements in 1962. Representatives of various sciences viz. geology, seismology, hydrology, geophysics, geodesy, oceanography, soil mechanics and mining take part in this subcommission. It's task is to study recent crustal movements, both horizontal and vertical. Therefore the subcommission planned the following activities:

- gathering all available information in order to obtain a picture of recent movements and of the relative position of the land surface and mean sea level,
- repeating measurements to study recent movements, and setting up new measurements in proper areas to fix the present situation for future comparison,
- co-operation with the International Commission on Recent Crustal Movements.

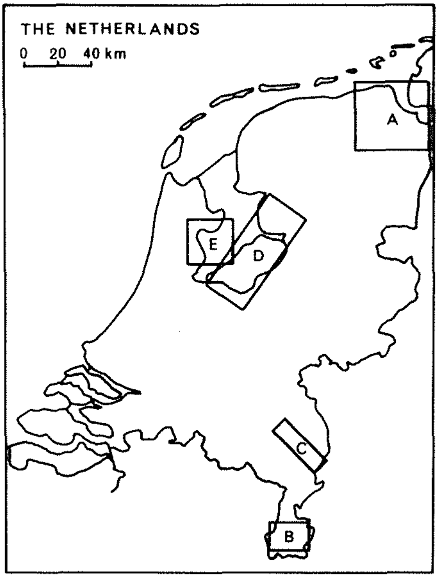

Fig. 7 Map showing locations where subsidence is studied.
A = Groningen gasfield
B = coal mining
C = Peel-boundary Fault
D = recently reclaimed polders
E = Markerwaard project

Waalewijn (1966) gives an enumeration of the points of interest of the subcommission, which is updated in the following lines. Fig. 7 shows the location of several of the sites mentioned.

In the Netherlands much attention always has been paid to the problems discussed here. As has been mentioned before relative sea level movements of the North Sea have been measured and studied since 1682. However the problem itself is not a regional, but a global one. Therefore the Geological Survey of the Netherlands initiated an international Sea Level Project, which was executed under auspices of the UNESCO. In the project called: "Sea-level movements during the last deglacial hemicycle (about 15.000 years)" more than 20 countries actively participated. This project stimulated the gathering of information additional to the many sea level data collected and evaluated by several scientists in the past. Van de Plassche (1982) describes the Netherlands effort in his thesis: Sea-level change and water-level movements in the Netherlands during the Holocene. His study results in a most probable mean sea level curve for the Netherlands.

The subcommission attaches great importance to the study of future developments in sea level movements. Since several years all over the world much attention has been paid to the increase of the concentration of carbon dioxide in the atmosphere and to the (resulting ?) melting of ice in Antarctica, possibly causing a worldwide change in sea level. The subcommission tries to formulate a national project to study the sea level rise in the future 500 years.

The subcommission also pays attention to tectonic movements. The first order levellings intended to study tectonic movements over the country as a whole have already been mentioned. Besides local tectonic movements are investigated. In the south-eastern part of the country some important underground faults are known. Some levelling lines crossing a fault line have been measured several times. To investigate horizontal shift of the Peel-boundary Fault a network of stable points will be installed. The situation of which will be fixed by very accurate distance and angular measurements.

In the Netherlands a number of saltdomes are known. The subcommission initiated to start measurements of heights on and around several of these saltdomes not being exploited at that time.

Approximately half of the surface of the Netherlands is covered by Holocene sediments; their compaction can be considerable which make dikes very vulnerable in these regions. In recently reclaimed polders landsubsidences of not less than about 1.50 m may be expected to occur in the first century after reclamation. In this symposium several papers are devoted to the problem resulting from the construction of polders.

Movements as a result of human activities in general occur in areas where coal, gas, salt, oil or water extracted. Coal mining over a period of about 75 years caused subsidences of more than 10 m. Production of gas in the very large Groningen gasfield which started 20 years ago is expected to result in surface subsidence of more than 25 cm in the far future. In both regions subsidence is monitored by repeated levellings as well as gravity measurements. Problems related to mining activities are discussed in a paper presented by Pöttgens in this symposium.

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MECHANISMS OF COLLAPSE CONTROLLED SUBSIDENCE

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Abstract

Sink hole formation in Karstic lime stone is one of the widely known mechanism of sudden land subsidence. Its scale is large, distribution world wide, and action quite dramatic and as such it has attracted serious study by geologists and geotechnical engineers.

Not so drastic in scale, though equally hazardous, are processes of collapse in structurally unstable soils. Some of these mechanisms are associated with changes of water table in built up areas. Collapse induced deformations produce harmful differential settlements for buildings and earth structures.

In this paper various mechanisms and factors controlling magnitude and rate of collapse are reviewed. Soil gradation, initial degree of saturation, pore fluid chemistry, and applied stress level control the magnitude and rate of collapse on wetting. Double oedometer and direct shear tests are used to investigate the collapse behaviour of structurally unstable soils.

Introduction

Wide spread occurrence of localized soil subsidence due to subsurface tunnelling and piping has been reported from many parts of the world in both dispersive and Collapsible Soils (Ingles and Aitchison, 1973, Sherard, et.al., 1972, Aitchison, 1973). The type of soils involved vary from alluvial, colluvial, aeolian, and residual deposits. Aitchison (1973) grouped soils exhibiting collapse, under a broad category termed as structurally unstable soils. Yudhbir (1982) reviewed causative factors and mechanisms of collapse in residual soils. Nature and mechanisms of wide spread collapse in Loessial Soils in Romania were presented at the 7th ICSMFE (1969) during speciality session No. 5 - Engineering Properties of Loess. In loessial deposits in Romania collapse on wetting, both under self weight and under additional stress, have been reported while in case of residual soils collapse occurs on saturation under stress only (Yudhbir, 1982). Limited test results (Yudhbir, 1982) indicate that dispersive soils could be collapsible too but reverse is not always true.

Ingles and Aitchison (1969) illustrated changes in existing soil-water system as the main causative mechanism of localised tunnelling and subsidence. This rather broad mechanism encompasses - (1) Changes in matrix suction upon wetting of partially saturated soils and (2) lessening of interparticle bonds in both clay and salt bonded soils. It is interesting to observe that causes of triggering mechanisms and eventual collapse in karst lime stones are similar to those stated above in case of soils; viz., changes in ground and surface waters (1) either a lack of water (depression of ground water table) or (2) an abundance of water (diversion and concentration of surface water flow) can enlarge a subsurface solution void and cause the collapse to occur (Benson, 1978). In both cases local subsidence results from disturbance of existing hydrologic regime.

Sherard et.al (1972) have described a mechanism of piping failure in dispersive agricultural land (FIG.1). Percolation of rain water through drying cracks results in internal erosion of dispersive clays which eventually leads to collapse of unsupported roof producing tunnels and sink-hole like features (see Ingles and Aitchison, 1969, Sherard et al. 1972, and Sherard and Decker 1976 for cases of local subsidence in embankment dams, slopes and cuttings involving dispersive soils).

In this paper various mechanisms of collapse controlled subsidence in structurally unstable soils are reviewed. Factors controlling collapse and the relevance of double oedometer and direct shear tests to evaluate collapse potential of geologic materials is brought out.

Mechanisms of Collapse

In clay and salt bonded soils collapse may result from reduction in interparticle bonds due to the advent of water, with or without any additional disturbing force.

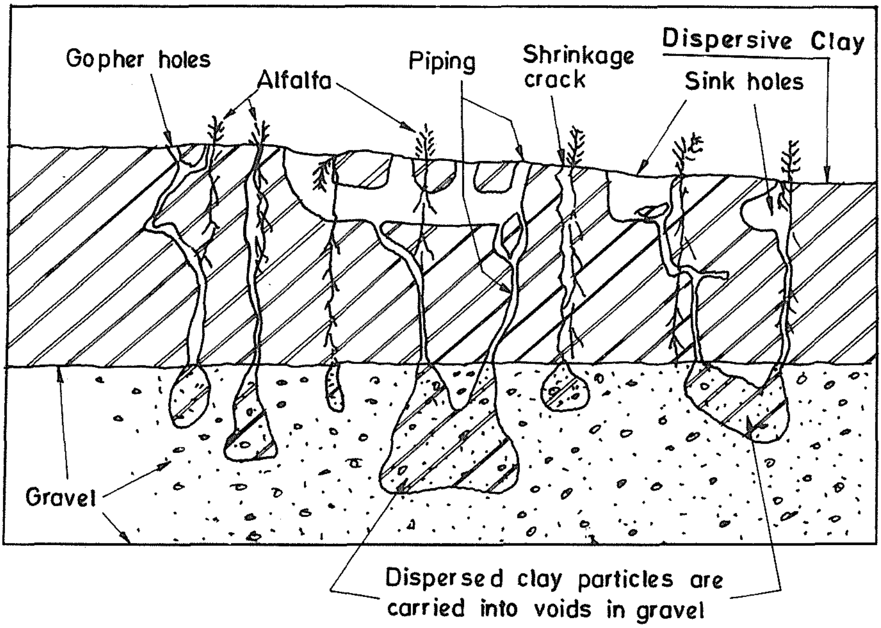


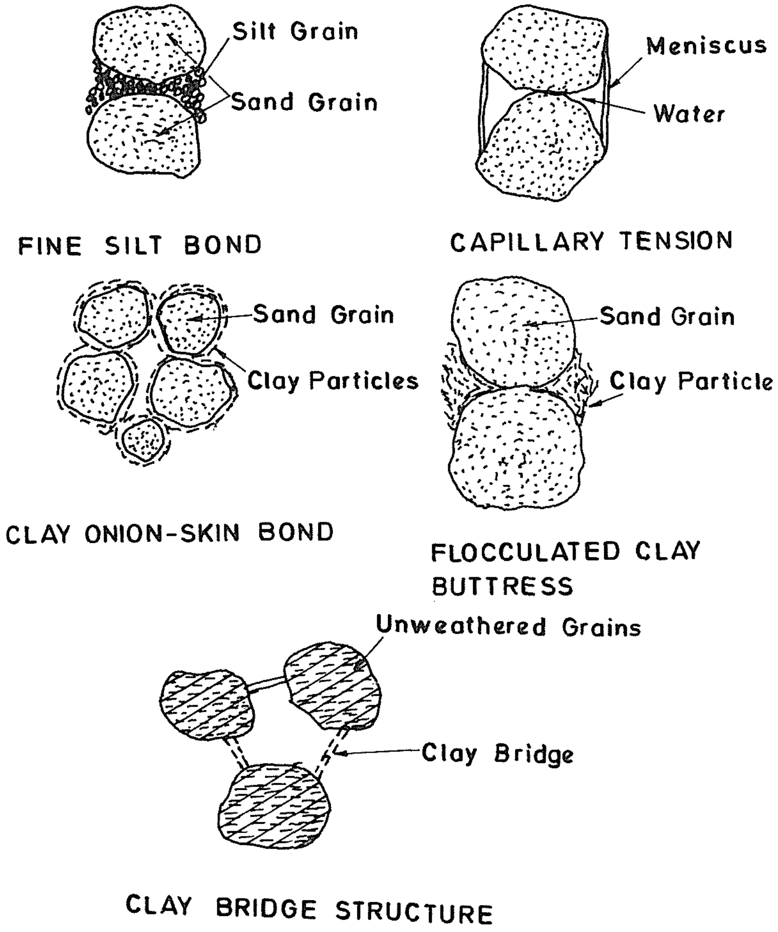
FIG.1 Sketch of collapse in dispersive soils

Collapse under self weight may result in arid area soils, such as; loess, clayey sands etc. in which the pores are wetted to full saturation for the first time. Gross dilution of salts in the electrolyte at the interparticle contacts (Ingles and Aitchison, 1969) and or gross reduction of matrix suction in these partially saturated soils (Booth 1975; Yudhbir, 1982) may be sufficient to cause substantial collapse of the land surface. Collapse controlled subsidence on wetting under self weight has been reported in thick Roamian Loess deposits (minimum thickness 6 to 8 m) in which cementing agents such as limonite and highly altered organic matter form water soluble bridges between aggregates. Deep loess profiles not showing any sensitivity to wetting under self weight only contained cementing agents as films covering the particles or filling the voids. Majority of soils however experience collapse on wetting under imposed stresses - the amount and rate of collapse would depend on the governing mechanism of interparticle bonding.

Fig. 2 shows some of the mechanisms proposed to explain collapse on wetting. Earliest mechanism of clay bridges between unweathered grains in open structure (Knight 1961), stipulates loss of strength of clay bridges on wetting which leads to subsidence due to collapse of open structure as soon as strength falls below the applied shear stresses. Buttress type clay bonds produce collapse on wetting under stress due to breakdown of buttress structure. The Onion-Skin effect, resulting from clay particles coating quartz grains, offers electro-chemical bonding strength to the open structure. On wetting the adsorbed water film thickness increases which results in decrease of bonding resistance and the open structure experiences collapse (Arman and Thornton, 1973). Temporary strength resulting from Silt-silt and Silt-sand bonds and capillary forces in partially saturated state decreases drastically on wetting and the open structure experiences collapse under stress.

The rate of collapse controlled settlement is fastest when governing mechanism is loss of strength on wetting. In case of clay bonds, depending on the impermeability of clay particles (controlled by the nature and amount of clay fraction present) collapse may be sudden or some times as much as 20-30 percent of total collapse may occur slowly as shown in FIG.3. In partially saturated structurally unstable soils, rate of collapse may be related to elimination of matrix suction which reduces apparent cohesion in these soils. The rate of elimination of matrix suction would depend on the rate of advance of wetting front which is time dependent and is controlled by the initial degree of saturation (Morgenstern and de Matos, 1975) In FIG.3 Lam Sam Lai soil is dispersive and non-collapsible under in-situ density and moisture state, while Khon Kaen soil is collapsible and non-dispersive. Lam Sam Lai soil shows collapse when compacted dry of optimum at a dry density of 1.73 t/m^3, (in-situ value of 2.03 t/m^3) but the rate of collapse is much slower compared to Khon Kaen soil, due to the fact that collapse in Lam Sam Lai soil is due to the dilution of pore fluid and in case of Khon Kaen soil it is because of elimination of matrix suction.

FIG.4 shows collapse on wetting under different stress levels as obtained from double oedometer test on weathered granite (see inset for loess). For Romanian Loess it has been found that at low stresses ($\leq$ 1 kg/cm^2), slow wetting gives large collapse than sudden flooding. At high stresses this difference disappears. Collapse vs normal effective stress and collapse vs initial degree of saturation relationships are shown in FIGS. 5 and 6 respectively.



FINE SILT BOND

CAPILLARY TENSION

CLAY ONION-SKIN BOND

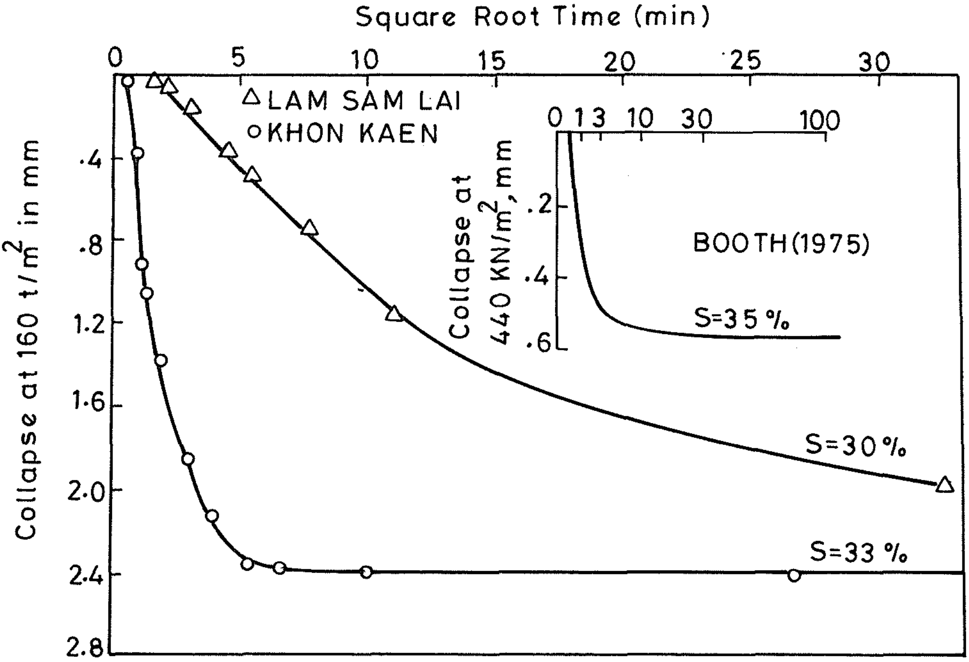
FLOCCULATED CLAY BUTTRESS

CLAY BRIDGE STRUCTURE

FIG.2 Types of Soil bonds and collapsible structures

The influence of initial dry density and degree of saturation on the value of stress at which maximum collapse occurs is well illustrated (FIGS. 5 and 6).

Gradation and consistency characteristics of soils exhibiting collapse on wetting under stress, are given in FIGS.7 and8 respectively. Result of double direct shear tests (FIG.9) indicate increase in strength for partially saturated soils upto a certain normal effective stress (controlled by initial suction) after which the failure envelope of in-situ state approaches that for initially saturated state and the two coincide at a higher stress. Examination of FIGS. 5 and 9



Lam Sam Lai Soil - Dispersive and noncollapsible under in-situ conditions

Khon Kaen Soil - Nondispersive and collapsible under in-situ conditions

FIG. 3 Rate of collapse

suggests that the value of critical stress at which maximum collapse occurs corresponds to the normal stress during shear at which shear strength is maximum (and so is dilatancy). Both collapse and rate of increase of strength decrease with increasing effective stress beyond the critical stress. This critical

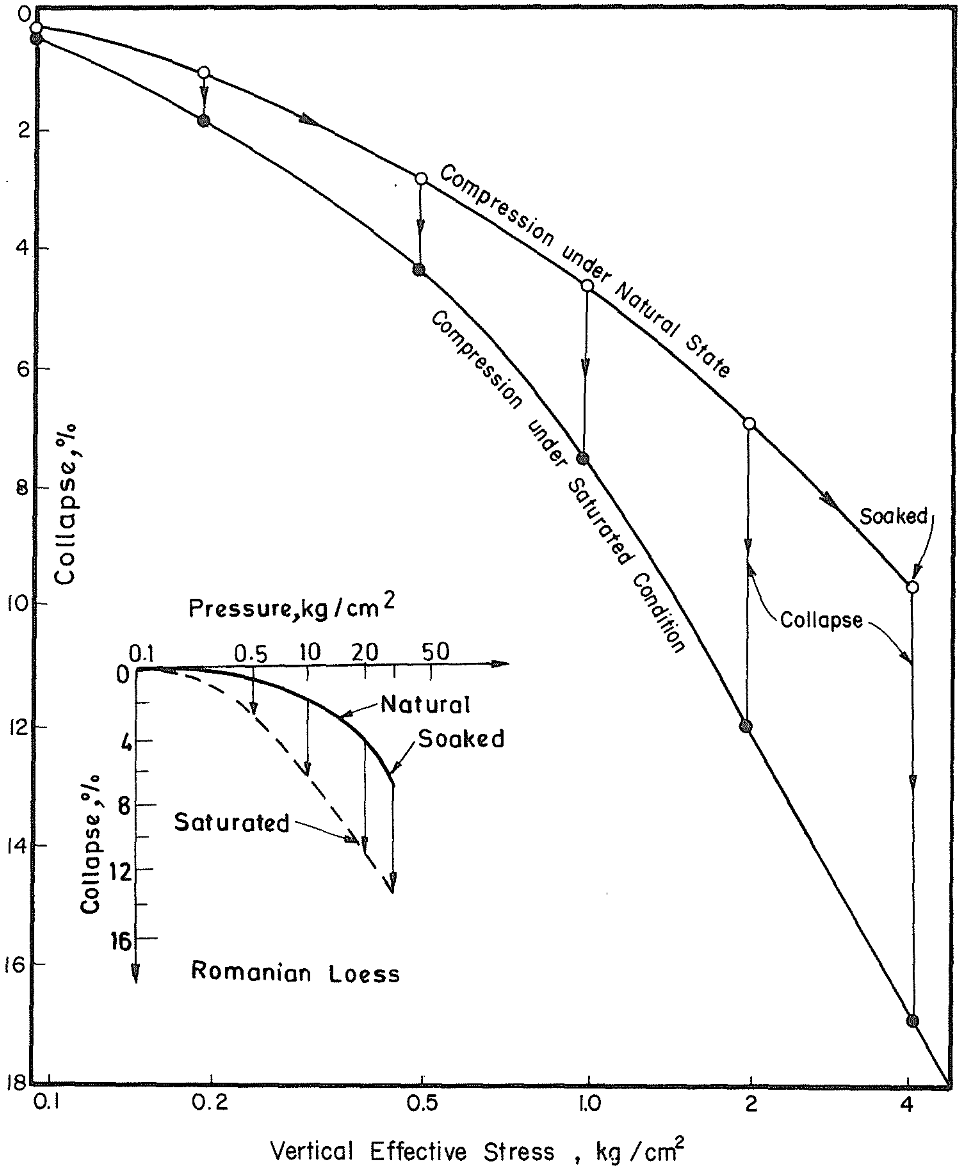


FIG. 4 Collapse on wetting

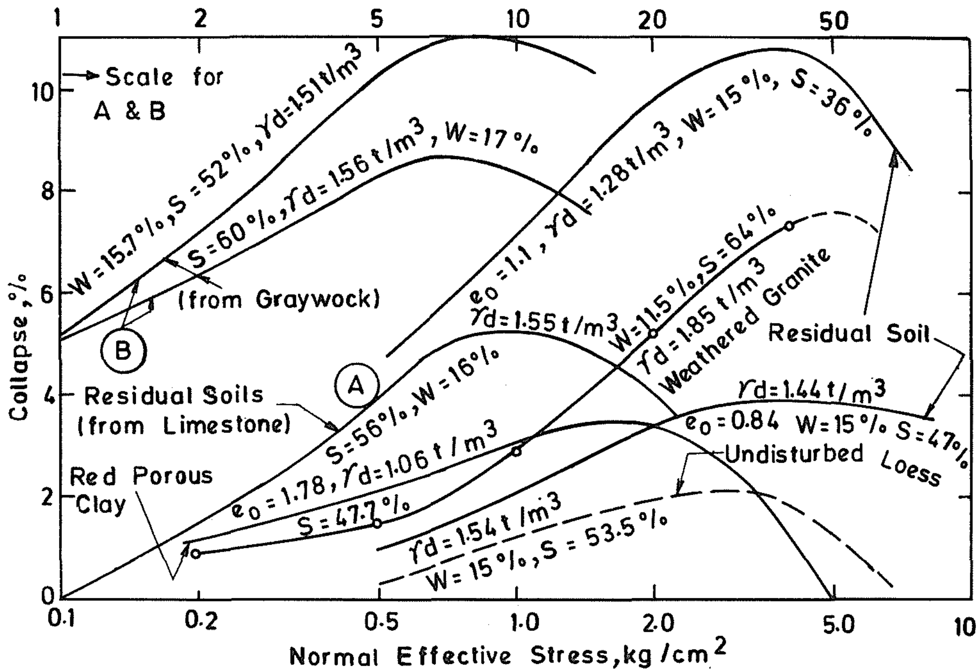


FIG. 5 Collapse vs effective stress level

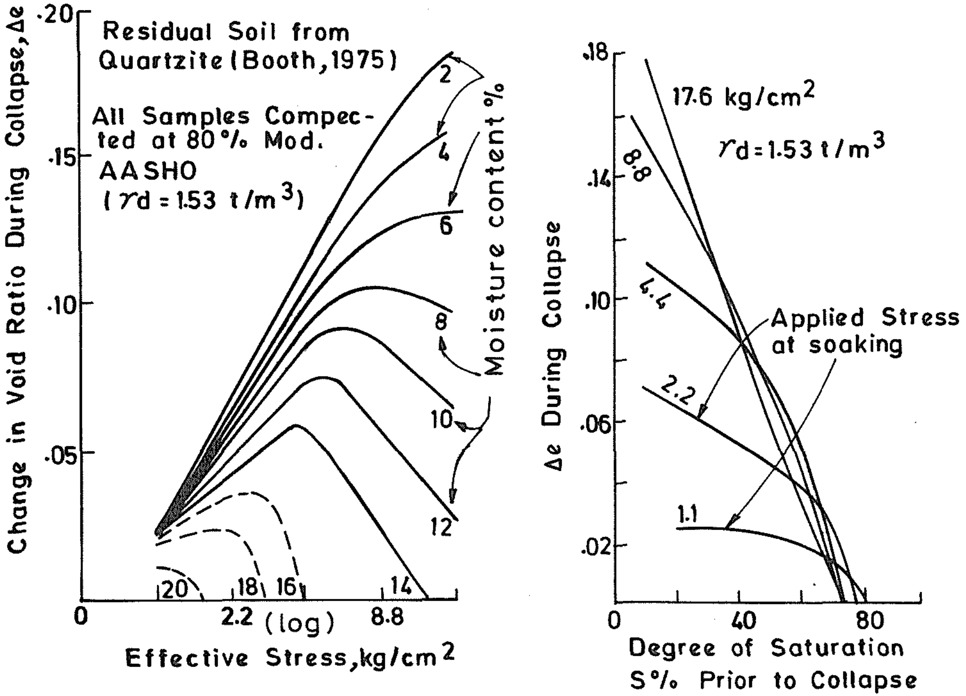


FIG. 6 Collapse vs initial degree of saturation

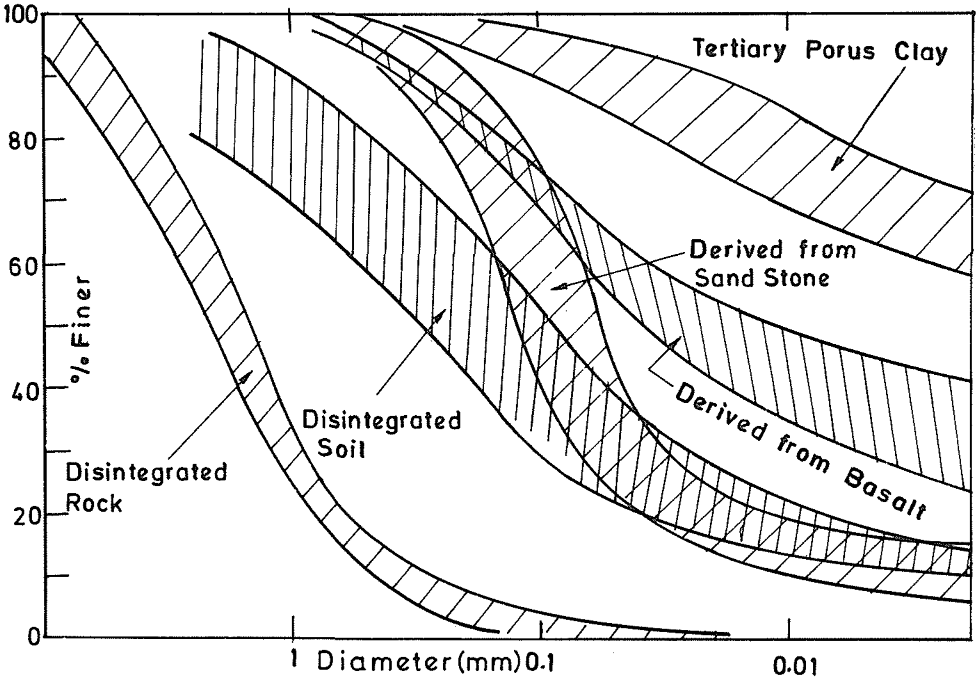


FIG. 7 Gradation of collapsible soils

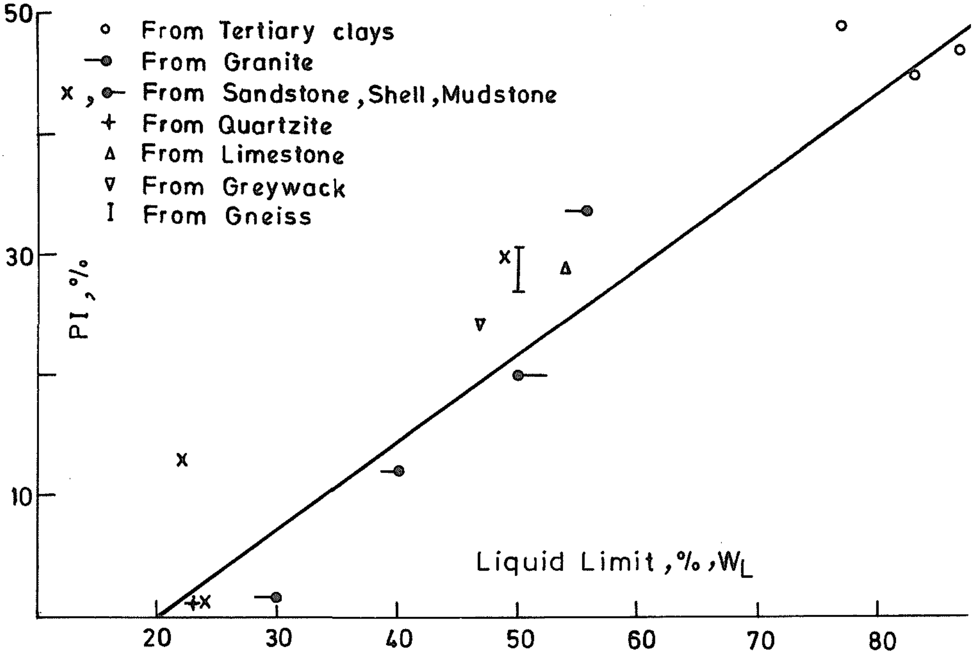


FIG. 8 Atterberg limits of collapsible soils

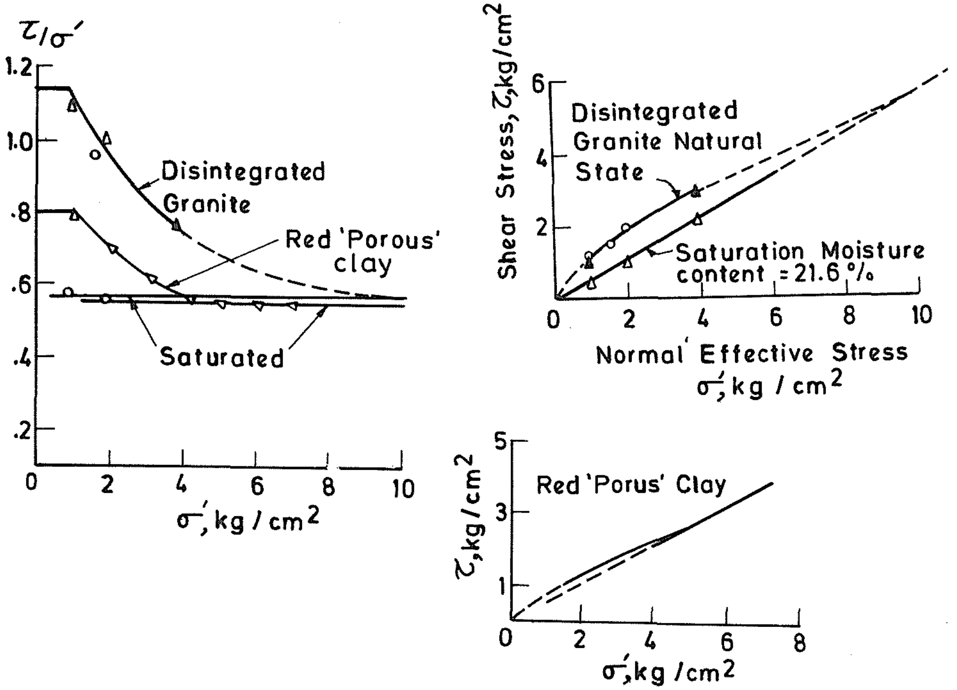


FIG.9 Double direct shear test results

stress corresponds to virtual preconsolidation stress (due to initial suction) and the maximum stress at which the in-situ failure envelope coincides with the initially saturated state failure envelope (FIG.9) determines the state when collapse on wetting under stress (FIG.4) becomes negligible.

Ingles and Aitchison (1969) have presented an excellent discussion on -(1) chemical origins of lessening bond strength and producing a metastable soil structure, and (2) conditions for producing hydrodynamic displacements which lead the metastable state to failure. They have also emphasized the static precondition for subsidence as the minimum voidage (not less than 21 percent). This condition may be met by the presence of shrinkage and drying cracks but unless the advent and passage of water is more rapid than the rate of swelling of soil along sides of cracks, no failure of soil can occur (Ingles and Aitchison, 1969). Sherard and Decker (1976) have presented detailed discussions on dispersive clays, related piping, and erosion in geotechnical projects. Only dispersive soils are not discussed in this paper.

Discussion

These considerations, based on the study of collapsible soils in double oedometer and direct shear tests tend to support the mechanism of collapse due to reduction in strength on wetting, and also suggest that collapse controlled subsidence is usually concentrated at shallow depths.

Subsidence due to groundwater with-drawal has been reported in many parts of the world. Much of this subsidence, without sharp surface discontinuities, is attributed to slow consolidation of sediments due to increased effective stresses arising from pumping overdraft. However, under certain geological settings, ground water lowering can either create conditions for collapse on subsequent inundation or precipitate immediate catastrophic collapse. Henkel (1982) has illustrated an example of catastrophic collapse resulting from groundwater lowering in dolomitic or limestone rocks in Far West Rand in South Africa. Wad, the highly compressible and insoluble residue of dolomites was underlying the weathered shales. Places where the upper overburden could arch across the lower compressible Wad, subsidence occurred without causing sharp discontinuities on the ground surface, however, places where benefitial arching effect was not adequate, catastrophic collapse took place.

Pixley Fissure in San Jaquin valley (California U.S.A.) developed at the edge of the Pixley subsidence bowl following heavy rains and flooding of this area on 26th February 1969. Guacci (1978) has attributed the formation of this fissure to the generation of extentional strains caused by the curvature of the ground surface in the vertical plane. He has further attempted to explain the enlargement of the initial crack (due to tension and not conspicuous before flooding) due to infiltration of water subsequent to heavey rainfall and flooding. It is tempting to suggest that the soils (oxidised sand silt mixtures with some clay) in the subsidence bowl were under compressive stresses produced by the pumping overdraft and and when surface waters percolated through the tensile cracks and flooded them, additional collapse induced subsidence took place which helped widen the existing cracks and produce the ground fissure observed on withdrawal of flood waters.

A 49 meter (m) high embankment dam founded on a collabsible sands and silts suffered 1 m collapse settlement at crest for a length of 1 kilometer (km) eventhough the compressible subsoils (void ratio, e = 0.6 to 0.9) were presoaked to minimise the consequences of collapse (Penman 1980). Yudhbir (1982) has described highly collapsible weathered granite foundations of a 2 km long earth dam (height varying from 10-15 m) in central India.

CONCLUSIONS

Various mechanisms of collapse controlled subsidence in wind-blown, alluvial and residual soil deposits have been discussed. Results of double oedometer and double direct shear tests are shown to be relevant laboratory tests to identify and evaluate collapse potential of geologic materials. Chemical, hydrological and geological settings conducive to collapse controlled localised subsidence have been briefly reviewed.

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GROUND ELEVATION LOSS IN THE RECLAIMED HULA SWAMP IN ISRAEL DURING THE PERIOD 1958 TO 1980

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Abstract

The Hula Lake and swamp in the north of Israel were drained and reclaimed for agricultural use in 1957. The newly exposed soils were mainly peats on the reclaimed swamps and lacustrine sediments with a high organic matter content in the reclaimed lake area.

Drainage and tillage of these soils initiated processes of consolidation and compaction of the soil, as well as desiccation and oxidation of the organic content, and removal by wind erosion of the topsoil pulverized by mechanized tillage equipment.

These processes have resulted in continuous lowering of the ground level, thus gradually increasing the frequency and extent of flooding of the reclaimed area. Moreover, the oxidation of the organic material yields considerable quantities of nitrates which are leached into the drainage network and carried into Lake Kinneret, Israel's main source of drinking water.

This paper discusses these processes and their environmental impact, as well as potential ways and means to reduce both the rate of elevation loss and the amounts of nitrates leached from the Hula into Lake Kinneret.

Introduction

The Hula Valley in the north of Israel forms the lower part of a catchment area roughly 10 times its own size. As a result of blockage of the valley outlet by a lava flow in ancient times, the outflow to Lake Kinneret (the Sea of Galilee) via the Jordan River was dammed up, resulting in extensive flooding of the valley. The valley floor was gradually built up through deposition of layers of erosion products from the surrounding catchment, alternating - according to the flooding history of the valley - with layers of autochthonous lacustrine deposits and peat, all of varying thicknesses.

Throughout known history the southern end of the valley was occupied by a small lake covering about 13 sq km. In the present century, this lake was surrounded by extensive swamps mainly on the northern side, covering an area of 40 to 60 sq km, depending on the season. The vegetation of these swamps consisted mainly of Cyperus papyrus, Polygonum accuminatum, Phragmitetum communis, Scirpeto, etc.

In the early fifties work started on the drainage of the Hula Valley, with the dual purpose of eradicating the endemic malaria plaguing the region and reclaiming the lake and swamp areas for agricultural use. The drainage was accomplished by deepening the Jordan River bed at the outlet from the valley, and work was completed in 1957.

The soils exposed by the receding water were mainly peats, in the reclaimed swamp, and lacustrine sediments in the reclaimed lake area. The peats contained 30 to 60% organic matter with a C:N ratio of 10 to 15 and a

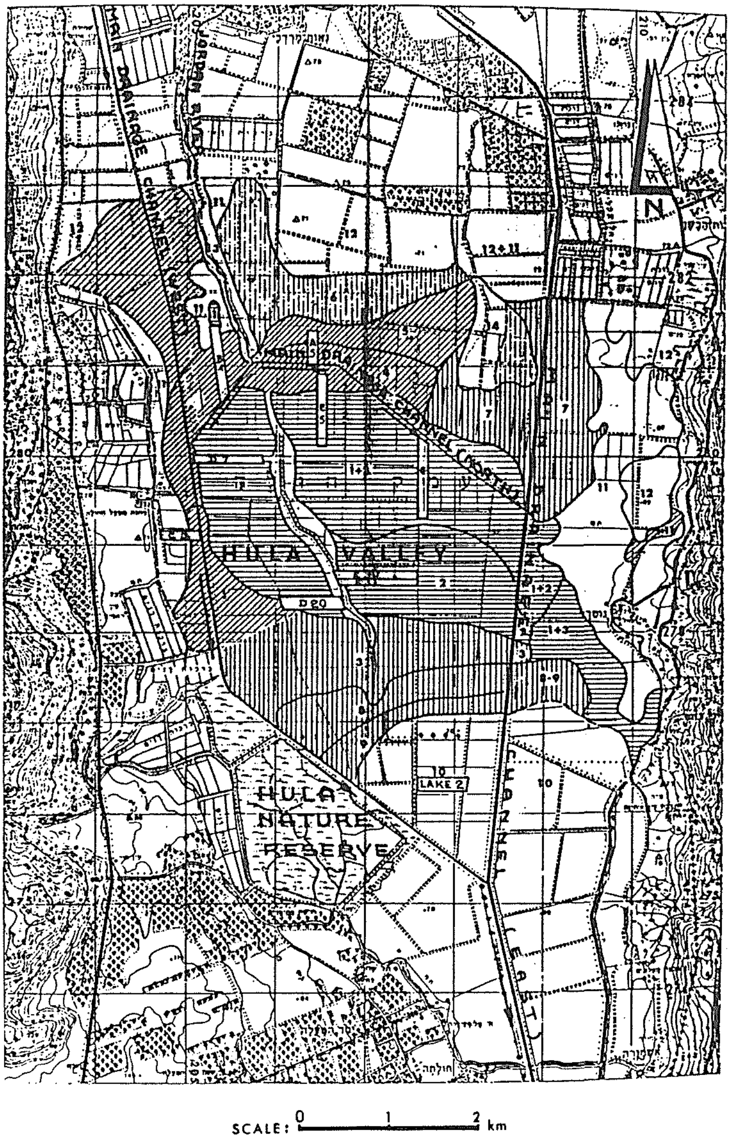

Figure 1

Location map of the observation plots

pH ranging from 5 to 7.5. The lacustrine sediments contained about 10% organic matter and 60 to 80% calcium carbonate. Their pH was about 7 to 7.5 and their structure very porous and friable.

Cultivation of the drained lands started in early 1958. The irrigation system introduced was modelled on that used on land of similar origin in the USA, and consists of a water distribution system made up of a dense network of 1-m deep channels and ditches surrounding the plots. To wet the crop root zone, water is let into these channels, thus raising the groundwater table over wide areas. The groundwater table is controlled by a system of 4" diameter mole drains constructed at right angles to the distribution-cum-drainage ditches, at a depth of about 0.5 m. The mole drains, made by dragging a 4" diameter bullet-like device behind a crawler tractor, are spaced at intervals of 6 m, and also serve the twofold purpose of irrigation and drainage.

Under this system the channel and ditch network must be cleaned at least once every season, and the mole drains must be totally renewed at least once a year.

The subsidence of organic soils following drainage, reclamation and cultivation is a well known phenomenon, and its occurrence in the Hula reclamation project was foreseen. To monitor the rate of the process, the surface elevation on six observation plots located in the former swamp area was determined in 1958 shortly after completion of the drainage operations, and the measurements were repeated in 1964, 1965, 1970 and 1980. In 1965 four additional observation plots were established - three on the reclaimed swamp, bringing the total there to nine, and one plot on the reclaimed lake area.

The monitoring of the ground levels of these 10 plots over a period of about two decades confirmed the occurrence of a continuous elevation loss. The observations are presented in this report. The causes of the phenomenon are discussed, as well as its consequences, environmental impact, and possible ways of reducing the rate and detrimental effects of the process.

Methods

Ground surface elevations were measured at the points of intersection of a 30 m by 30 m grid on each of ten plots established for this purpose. The 9 plots in the reclaimed swamp are representative of about 40 sq.km of peat soils, and the plot in the reclaimed lake area is representative of about 13 sq.km of lacustrine deposits. See Fig. 1.

The elevations were tied to the Israel Geodetic Survey benchmarks established by precise levelling on the slopes of the mountain range to the west of the Hula swamp, on shallow terra rossa soils underlain by limestone rock. A detailed description of the surveying work and the method employed to estimate the elevation losses are given in Refs. 2, 6 and 7.

Results and Discussion

The elevation losses on the ten plots are presented in Tables 1 and 2 and Figs. 2 and 3.

Three points may be noted from the results:

- The process is continuous for both the peat soils and lacustrine deposits
- The rate of the process is considerably higher in the peat soils
- Though the rate of the process on individual plots may differ considerably over short periods, the overall rate of elevation loss over the long term is remarkably uniform. This is in line with what is known from experience elsewhere (5,8,9,10), namely that the process of elevation loss of drained peat soils continues as long as cultivation continues and the organic material in the soil is not completely exhausted.

The elevation loss in the reclaimed Hula swamp and lake is the result of

TABLE 1: MEASURED SURFACE ELEVATIONS DURING THE PERIOD 1958-1980

Plot	Area (ha)	Elevation (m)[1)			
		1958	1965	1970	1980
A5	3.24	66.88	66.11	65.82	64.90
B4	7.02	67.19	66.56	66.23	65.53
C3	3.51	66.71	66.12	66.08	65.12
D7	6.75	66.50	65.81	65.24	64.83
D20	5.94	65.76	64.83	64.40	63.78
E5	7.02	66.64	66.07	65.68	64.95
E30	5.40		64.98	64.66	63.86
E31	5.22		65.10	64.73	63.91
Z1	7.56		65.16	64.81	64.05
Lake	8.10		63.71	63.52	63.28

1) The elevations are mean averages calculated from the surveyed elevations of a 30 by 30 meter grid.

TABLE 2: MEAN AVERAGE YEARLY SURFACE ELEVATION LOSS (cm)

	Period				
Plot	1958-65	1965-70	1970-80	1958-80	1965-80
Peat Area:					
A5	11.0	5.8	9.2	9.0	8.0
B4	9.0	6.6	7.0	7.5	6.8
C3	8.4	0.8	9.6	7.2	6.7
D7	9.9	11.4	4.1	7.6	6.5
D20	13.3	8.6	6.2	9.0	7.0
E5	8.1	7.8	7.3	5.1	7.5
E30		6.4	8.0		7.5
E31		7.4	8.2		7.9
Z1		7.0	7.6		7.4
Mean for peat area:	10.0	6.9	7.5	7.6	7.3
Lake		3.8	2.4		2.9

subsidence processes as well as erosion. The processes involved are the following:

- Consolidation of the sub-base due to removal of buoyancy forces as a result of the drainage
- Desiccation of the organic fabric of the soil in the drained and aerated part of the profile
- Oxidation of the organic material in the drained and aerated zone of the soil

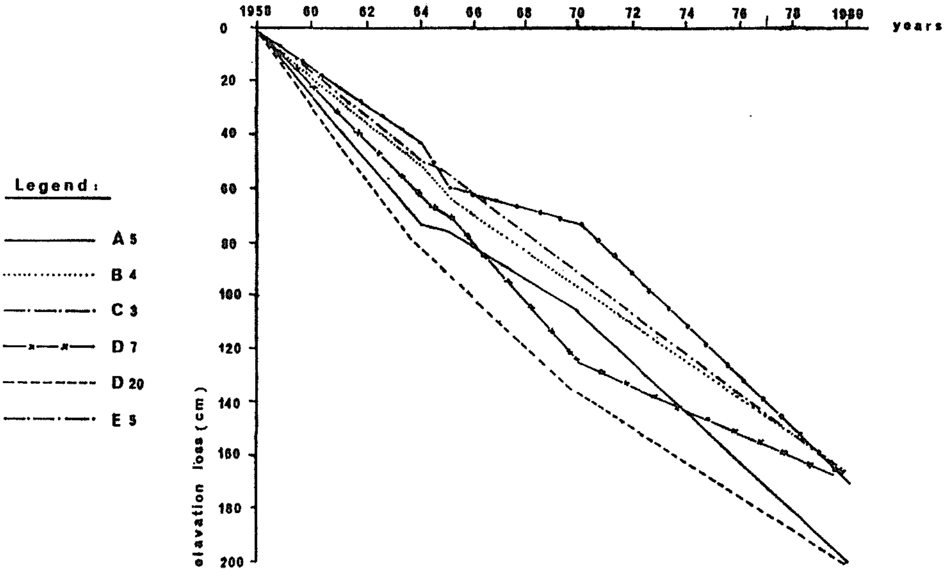

Figure 2

Accumulated elevation loss in the surveyed plots of the Hula peat soil

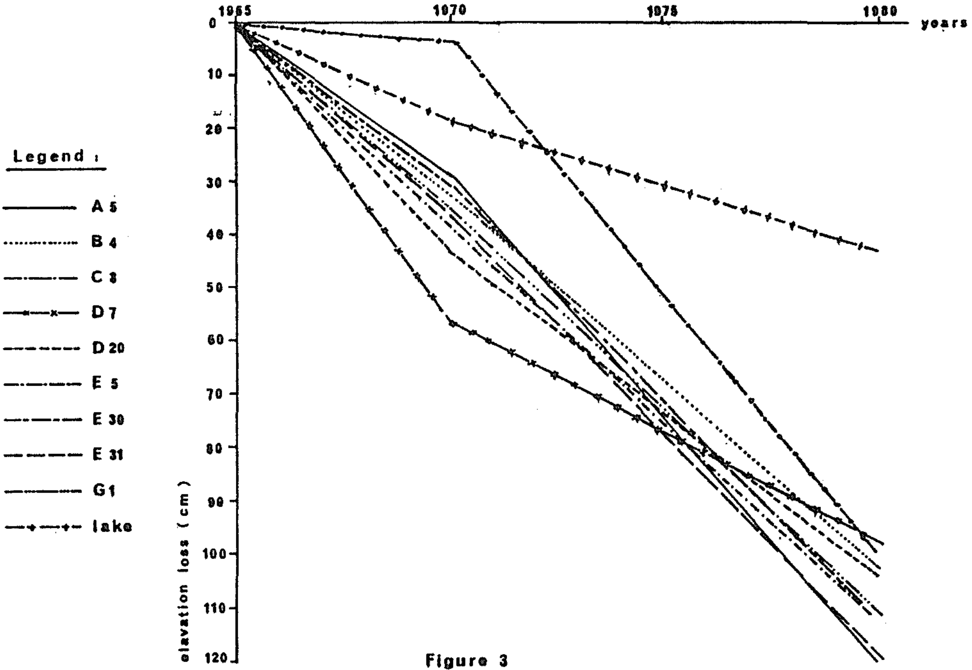

Figure 3

Accumulated elevation loss in the surveyed plots of the drained Hula peat and lake soils

- Compaction by mechanized agricultural equipment.
- Removal by wind erosion of the topsoil pulverized by mechanized agricultural equipment.

The lower rate of elevation loss in the reclaimed lake compared with the swamp can be attributed to the fact that the lake soils have higher soil particle density, lower moisture capacity, and lower organic material content, which leads to less shrinkage resulting from desiccation and oxidation of organic matter.

The above processes have had an important impact on the ecology of the area. The main effects are the following:

- The progressive lowering of the reclaimed area relative to the outlet from the valley has increased the frequency, extent and duration of flooding in these areas. This has necessitated repeated works to increase the carrying capacity of the main drainage channels.
- Oxidation of the organic material in the drained and aerated zone of the soil yields considerable quantities of nitrates, which are carried away by the drainage effluent and finally reach Lake Kinneret (1, 3, 4).

Monitoring of the nitrate flux to Lake Kinneret indicates that the amounts carried into the lake from the reclaimed Hula lands constitute the largest single source of nitrate flow into the lake. The potential damage to the water quality of Lake Kinneret, Israel's main source of drinking water, is by far the most serious hazard resulting from the ongoing and unchecked processes described above.

A clue to finding economically feasible means of inhibiting the harmful processes set in motion by the drainage of the Hula Valley may be found in the occasional deviations from a uniform rate of elevation loss discerned in some of the observation plots for short periods (see Fig. 2 and 3). These deviations are strongly correlated with local differences in agricultural management. While for most of the time the plots concerned, like the other plots, were sown to seasonal field crops and left fallow for almost half the year, deviations occurred when the crops raised were perennial forage crops. These crops protect the ground since they require less mechanical tillage and thus reduce the pulverization of the topsoil, and they provide a permanent ground cover which gives protection against wind and water erosion the whole year round (1,4).

Research carried out to follow up these observations indicates that perennial grasses like alfalfa, Rhodes grass or panicum also reduce oxidation of the organic material and remove, in the crop carried off the field, the major part of the residual oxidation products. Growing these crops will thus check further ecological damage.

Another potentially effective means of controlling the damage, also supported by research (1,4), is the use of sprinkler irrigation instead of the sub-irrigation mode described above. Under the latter method the irrigation-cum-drainage channels are cleaned each year and deepened by about 0.1 m to maintain their required level below ground surface. Thus more of the soil profile is drained each year and exposed to consolidation forces as well as to aeration and oxidation. Moreover, lowering the groundwater surface after irrigation acts like a bellows, drawing into the soil fresh, oxygen-laden air, thus regenerating the oxidation process. So the processes causing elevation loss are reinforced each year. Sprinkler irrigation, in contrast, besides preventing all this, brings about regular leaching of nitrates formed in the upper aerated layers of the soil, washing them into the lower anaerobic layers where denitrification processes prevail. Thus, irrigating with sprinklers may be expected both to reduce the rate of oxidation, which is an important factor causing elevation loss. and to reduce the nitrate residuals carried by flood-return flows into Lake Kinneret.

It remains to be shown by controlled experiments on commercial plots, that these methods are indeed effective and at the same time economically feasible.

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SUBSIDENCE IN THE RECENTLY RECLAIMED IJSSELMEERPOLDER "FLEVOLAND"

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Abstract

The surface of Flevoland (970 km^2) reclaimed from the bottom of the IJsselmeer (lake) is liable to subsidence, resulting from physical ripening (top soil; circ. 1.5 m) and compaction (subsoil) of the soft to very soft sediments (porosity 65-90%) after drainage. The polder is subdivided into two parts, emerged in 1957 and 1968. Level records have been taken at eighty permanent measuring sites annually. The regional pattern of the subsidence in 1983 (0.12-1.02 m) is given (Fig. 3). It is related to the thickness of the soft layer (Fig. 1) and its composition (Fig. 2, Fig. 4). The mean annual subsidence of each part appears to be rectilinearly proportional to the logarithm of time (Fig. 5). For about the first ten years the subsidence rate is relatively low as compared with this rate afterwards, mainly due to the initially poor, but gradually improving drainage conditions. The observed relations enable to predict subsidence by extrapolation and to check afterwards the predictions made before reclamation.

Introduction

In the centre of the Netherlands an extensive artificial lake (IJsselmeer; initially 3,740 km^2) is found. In this lake from 1930 on four polders with a total area of 1,650 km^2 have been endiked and reclaimed.

The surface of the polders subsides remarkably in the course of time. In vast parts the expected subsidence ranges from a mere 0.10 m to over 1.25 m in the first century after emergence, being on average about 0.90 m. This subsidence has an important impact on the water marks to be installed. For many constructions this water mark is decisive. Hence, it is important to have a prediction of the subsidence as accurate as possible.

In the two youngest polders, Oostelijk Flevoland (part I; emerged 1957) and Zuidelijk Flevoland (part II; emerged 1968), initially at some ninety permanent measuring sites the subsidence is recorded by taking the level in spring every year.

Geological conditions

The profile consists of a layer of mainly soft to very soft holocene sediments (mineral) and sedentates (peat), overlying a sandy pleistocene subsoil. No compaction is found in these sandy sediments, so they do not contribute to the subsidence of the surface. The thickness of the holocene layer ranges from a few decimetres to about 6 m and incidentally to some 8 m. The thickness of the Holocene is shown in Fig. 1.

In the Holocene a number of layers occur, deposited or formed (peat) in various subphases of the Holocene under varying environmental conditions, resulting in variations in texture and softness. In Fig. 2 a schematic picture of the sequence of the various deposits is given. Not all layers occur everywhere, being either not deposited or eroded later on. The last in particular happened often to peat.

Texture and initial consistency

The bottommost peat layer is rather compact. The organic matter content is in the range of 60% and the water content is around 350 to 400%. The porosity is in the range of 80%.

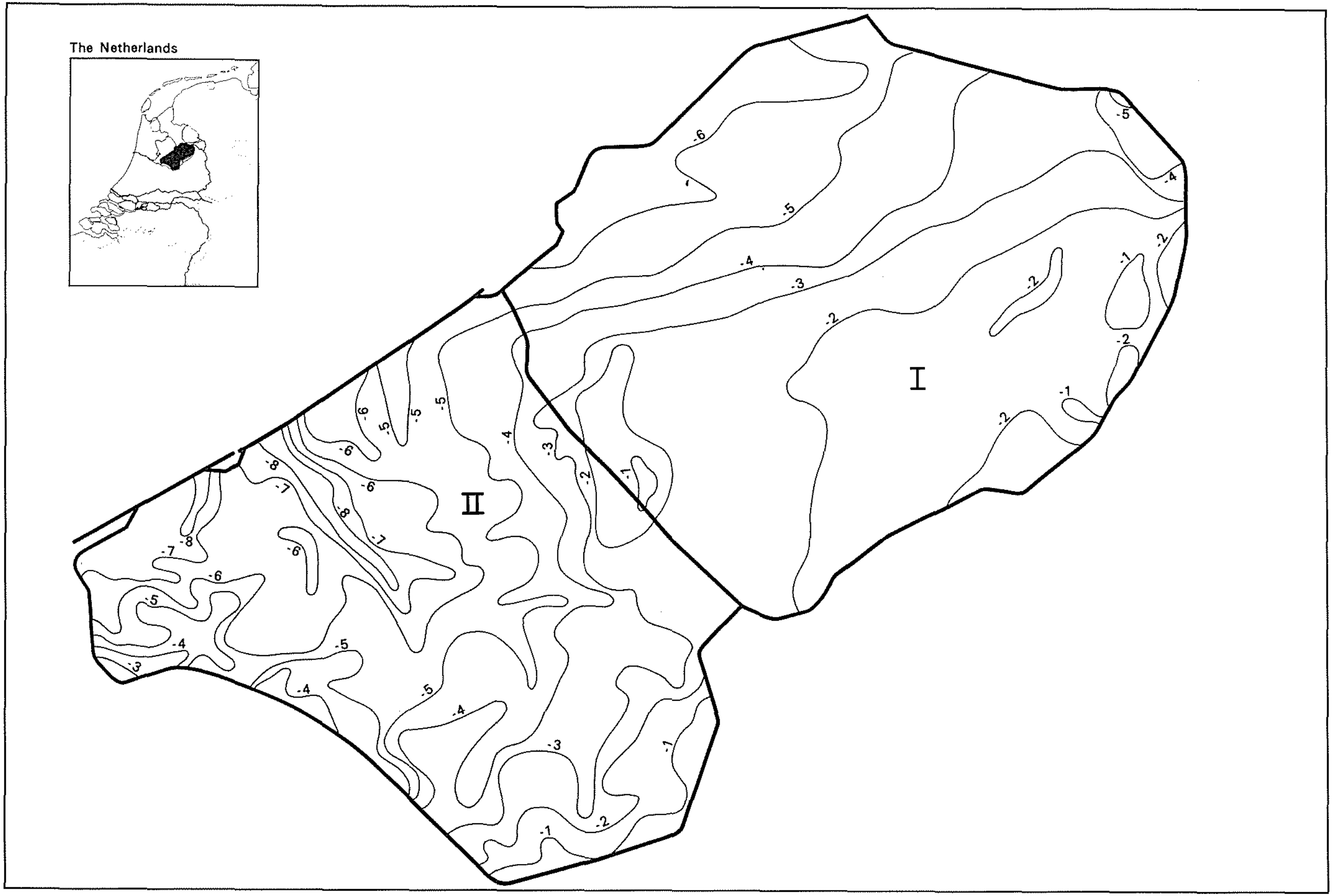

Fig. 1. Thickness of the holocene sediments before emergence (simplified; scale circ. 1:275,000)

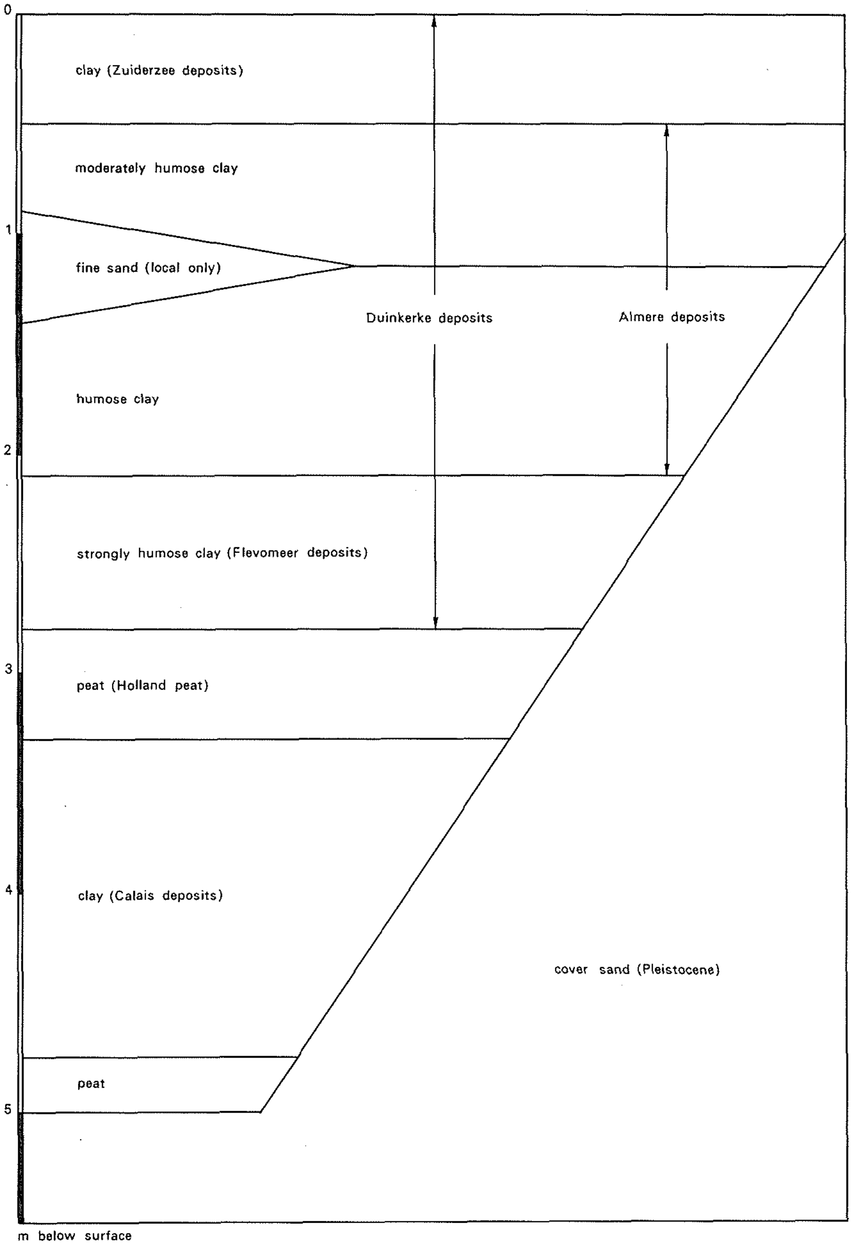

Fig. 2. Schematic profile of the holocene sequence

The Calais deposits consist of a drowned system of gullies with the adjacent levees and backswamps. The levees are relatively firm. The clay contents are in the range of 25% and the porosity is about 65%. The relatively low lying backswamps are soft with a porosity in the range of 70 to 75% at clay contents of 30 to 40%.

On top of the Calais deposits peat was formed again (Holland peat). This peat is supposed to have had a thickness of 2 to 3 m, but later on a lot of it has disappeared by erosion. The organic matter contents are mainly above 75 to 80%. The water contents are 600 to 800%. The resulting porosity ranges from 85 to 90%.

In this peat area from 1000 BC on a number of smaller and larger lakes was present. By erosion these lakes enlarged gradually, eroding the Holland peat. The eroded peat was deposited at the bottom of these lakes. This subaqueous sediment (Flevomeer deposit) is clayey (20%) and rich in organic matter (20 to 25%). Its porosity is about 80%.

From AD on gradually a better connection between the enlarging lakes and the North Sea arose, resulting in an increasing supply of mineral matter. The erosion of the peat continued, but at a decreasing rate. In the last part of this period man started to defend his land by embankments (1000 to 1200 AD), also bringing about less erosion. So the bottommost parts of these subaqueous Almere-deposits are rich in organic matter (circ. 15%) and the topmost parts relatively poor (circ. 5%). The clay contents are in the range of 20% (bottommost parts) to 30 to 35% (topmost parts). During a part of this period in the north-west sandy sediments were deposited (3 to 5% of clay). The porosity of the Almere-deposits ranges widely, from 45% (sandy) to over 80% (humose to very humose clay).

In the 16th century the erosion of peat had become very minor. The more and more improving connection with the North Sea brought about an increasing supply of mineral matter. From that time to 1932 a marine, subaqueous sediment (Zuiderzee deposit) was deposited, ranging in clay content from a few percent to about 40%. The organic matter content is about 10% of the clay content. The porosity ranges from 50% (sandy) to 80% (rich in clay).

Installation of the permanent measuring sites and measurements

The permanent measuring sites have to be installed already before a polder is pumped dry. In both parts this has been done in the last summer before emergence. It has to be done so early, as a number of sites will be inaccessible most likely in the first year after emergence, due to the extremely low bearing capacity. However, subsidence progresses meanwhile and the initial level would be unknown.

The location of the sites is selected on the base of the data of the under water soil survey. The selection has been such, that on the one hand most profile types are comprised and on the other hand the grid is as regular as possible. The last was more succesfull in part II than in part I. The location of the sites is indicated in Fig. 3. Out of the original number of 94 sites 15 sites got lost for various reasons in the course of time.

At installation in part I an iron pipe (Ø 1") is driven from a ship about 1 m down into the sandy, non-subsiding pleistocene subsoil. In part II this was a plastic pipe (Ø 1"). The pipes end just above the lake bottom. The sites were marked by a concrete slab (Ø 1.0 m). Shortly before emergence, the sites have been marked by a long pole, as long, tall reeds cover the area within two or three years after emergence. The level of the lake bottom, being very flat, is sounded with a rod with enlarged foot (Ø 0.5 m) at some ten places around the ship, using the top of the iron or plastic pipe as a bench mark. The deviation of the mean value was only 0.02 m at the utmost. At the same time borings are made from the ship down to the

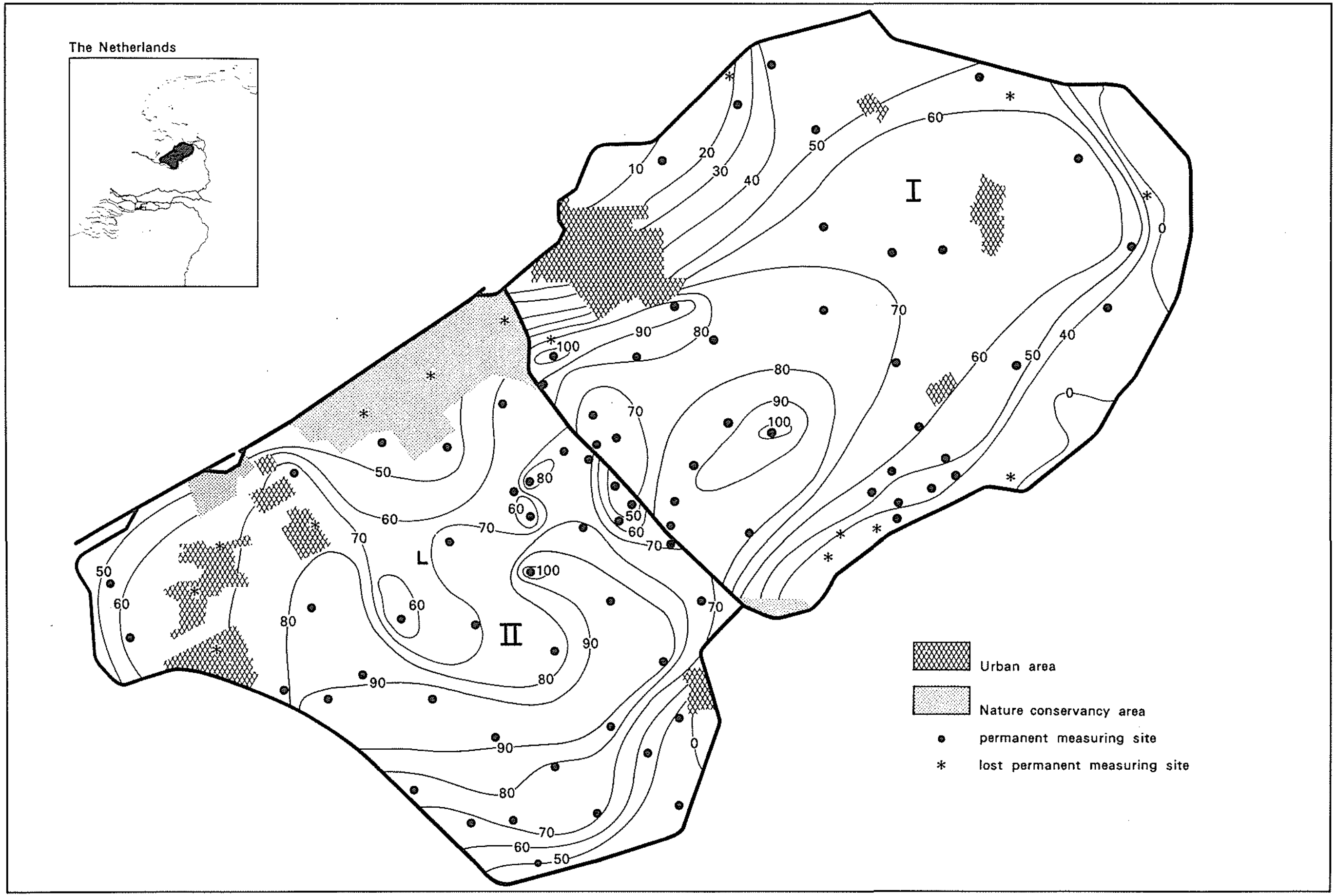

Fig. 3. Subsidence of the surface since emergence (part I 1957; part II 1968) until spring 1983 and the location of the permanent measuring sites (scale circ. 1:275,000)

pleistocene subsoil. Soil samples are taken from the sediments of the various geological phases of the Holocene.

The level of the sites is recorded again as soon as possible after emergence, using the top of the iron or plastic pipe as a bench mark. Afterwards, the level is taken every year in spring. The level per site is calculated as the mean of 20 records. Soil sampling is also repeated, initially every spring, but later on only once in three to five years, as the rate of the irreversible decrease of the water content slows down substantially in the course of time (De Glopper, 1973; Rijniersce, 1983).

Regional pattern of the subsidence in 1983

Part I of Flevoland was dry in May 1957, part II in May 1968. The subsidence from emergence until spring 1983 is shown in Fig. 3 by lines of equal subsidence. On average, the subsidence in part I is 0.64 m in 26 years and in part II 0.70 m, but in 15 years only. The greatest subsidence in part I is 1.02 m and in part II 1.01 m. The sites with the smallest subsidence show values of 0.12 m in part I and 0.30 m in part II. The lines in both parts of Flevoland do not fit at the separation dike between both parts, resulting from the difference in age between both parts.

In the topmost 1.0 to 1.5 m the shrinkage of the soil and therefore the subsidence of the surface is related to the clay content. An indication of the variations in the clay contents is given in Fig. 4 for the topmost 0.8 m. In part I the clay content decreases from the centre in northwestern direction. In the north-west below 0.3 to 0.8 m down to circ. 1.5 to 2.5 m a holocene layer of fine sand is present. This layer does not compact and has already compacted the underlying soft layers to a great extent before emergence. Therefore, the subsidence decreases remarkably in northwestern direction. From the centre to the southeastern dike the thickness of the Holocene decreases from roughly 3 to 4 m to less than 1 m (cf. Fig. 1). Also the clay content decreases in that direction, though not as much as to the north-west, apart from two areas where all the Holocene consists of sand. Next to this, in a zone along the southern half of this dike seepage occurs. This slackens the desiccation and, hence, the subsidence. Therefore, also the subsidence diminishes from the centre in southeastern direction. About half-way the separation dike between both parts of the polder the Holocene is also rather shallow, bringing about less subsidence.

The clay contents in part II are higher than in part I, so the subsidence is greater. There is no good relation between the subsidence and the clay contents as yet. Most probably the variations in time of actual reclamation play a more important part in the progress of the subsidence. Before the reclamation the drainage is poor and therefore the shrinkage slows down after a couple of years after emergence. Neither the irregular pattern of the thickness of the Holocene is expressed in variations of the subsidence so far, as the compaction of the deeper subsoil progresses very slowly. Only in the south the decrease in thickness of the Holocene is rather clearly shown in decreasing subsidence values.

Relation between subsidence and time

From soil mechanics it is known, that the secular subsidence of loaded, compressible layers is rectilinearly proportional to the logarithm of time. Considering the long period for which the subsidence in Flevoland will last, the process can be considered to be secular in nature. The total subsidence in the rural areas in Flevoland results from

- the shrinkage of the top soil (circ. 1.5 m), due to the irreversible drying, bringing about a contraction of the soil skeleton (the so called physical ripening), resulting from increased capillary forces in dry periods, mainly due to evaporation by the vegetation. This shrinkage and

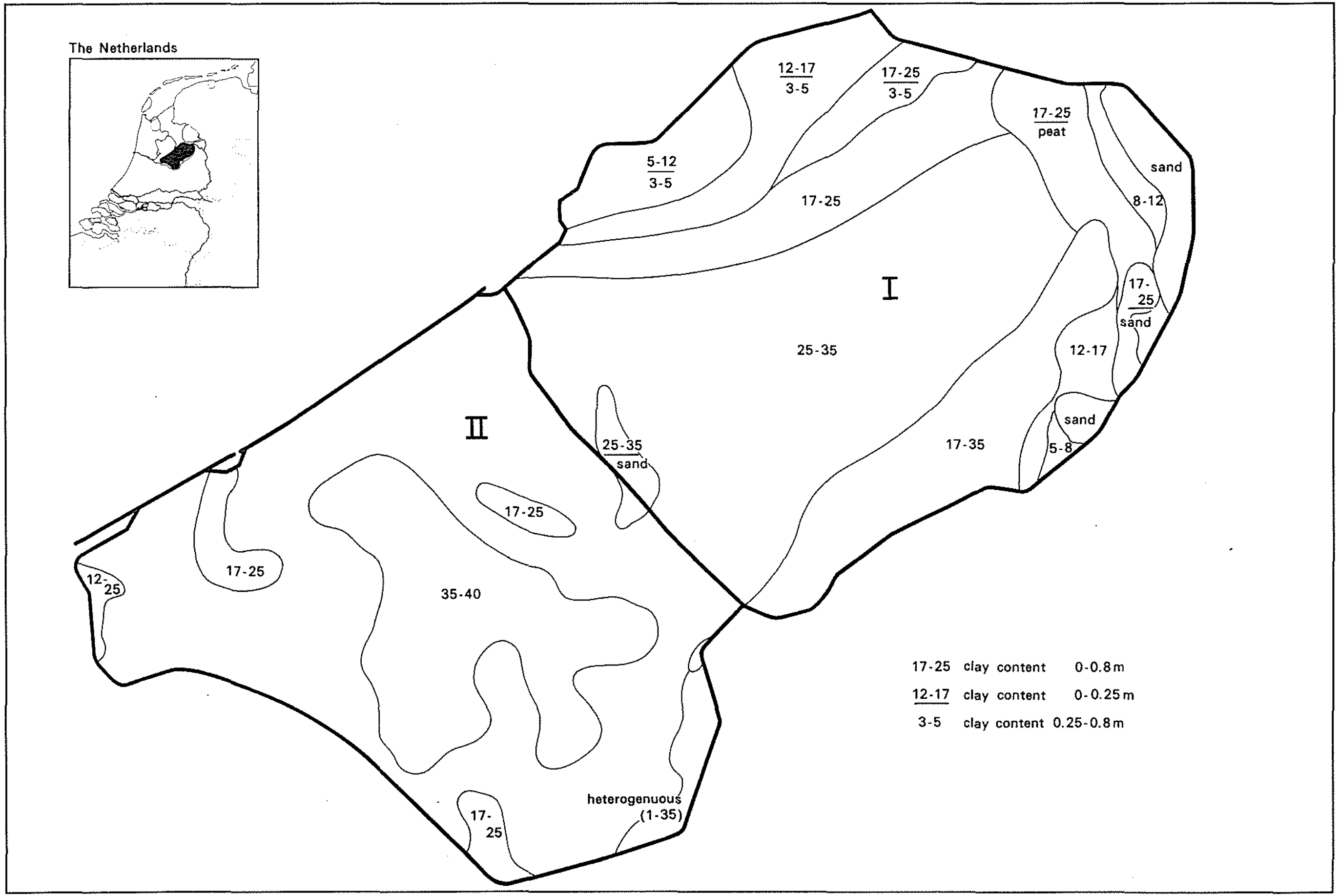

Fig. 4. Broad indication of the texture of the topmost 0.75 m of the Holocene (scale circ. 1:275,000)

the resulting subsidence is predicted by means of the density comparison method (De Glopper, 1969 and 1973) and
the compaction of the subsoil (below circ. 1.5 m) by squeezing of soil moisture, due to the increased load, resulting from the lowering of the groundwater table from in or above the surface to circ. 1.0 m below surface on average throughout the year. This compaction is predicted by means of the consolidation theory, according to the formulae of Terzaghi, Keverling Buisman and Koppejan.

Of course, both processes shift gradually from the one into the other at a depth of circ. 1.2 to 1.7 m. As the capillary forces vary widely throughout the year (from nearly zero in wet periods to 100 to 500 mbar (pF 2.0 to 2.7) in dry periods in young soils and 1,000 to 10,000 mbar (pF 3.0 to 4.0) in older soils; Rijniersce, 1983), it was not sure, whether the total subsidence should be also rectilinearly proportional to the logarithm of time.

The mean subsidence of all permanent measuring sites in part I and in part II was calculated from those sites still existing in spring 1983. The relation between this mean subsidence and the logarithm of time is shown in Fig. 5. In this figure also the evaporation surplus is given for the period May through September, being an indication of the drying effect of the summers. From October through April no or nearly no drying of the soil is found (precipitation exceeds evaporation) under Dutch climatic conditions. In this period the soil is completely rewetted to field capacity. The evaporation surplus in a given summer is indicated at the subsidence value of the next year, as the levellings are done in spring and the difference in level with the preceding spring results from the subsidence in the preceding summer. In autumn, winter and early spring (through April) nearly no subsidence, resulting from the physical ripening, occurs. The subsidence, resulting from the compaction of the subsoil progresses more or less continuously, but its contribution to the total subsidence is minor in general, being in the range of 5 to 15% out of the total.

In broad outline the relation between subsidence and time appears to be logarithmic in nature, though irregularities occur, related to the progress in drainage facilities and variations in the evaporation surplus.

With respect to drainage, not all the area of a new polder can be supplied with ditches, field ditches and the subsequent subsurface drainage system immediately after emergence. In the first ten years the drainage conditions improve gradually until all parcels are supplied with ditches and a subsurface system. In areas not yet supplied with ditches in spring a layer of water in the range of 0.05 to 0.15 m stays behind at the surface from the precipitation in winter. This results from the long distance the water has to cover and the low current velocities in the dense reed vegetation with a thick layer of died leaves at the soil surface. This water has to be evaporated at first, before the capillary forces can increase. Sometimes this water disappeared only very late in summer. At such conditions soil ripening and therefore subsidence is delayed and progress will be small.

The evaporation surplus varies widely from the one year to the other, ranging from 405 mm in 1959 to -25 mm in 1968. In the sixties the surplus was at the low side (mean 140 mm) and in the seventies at the high side (mean 225 mm), as compared with the mean value for the period 1958-1982 (190 mm). From 1980 through 1982 the surplus was around or a little below this mean value. The evapotranspiration by the vegetation is smaller than the evaporation from open water, at least by arable crops and grass. The evapotranspiration by a well developed, dense reed vegetation approximates the evaporation from open water rather close.

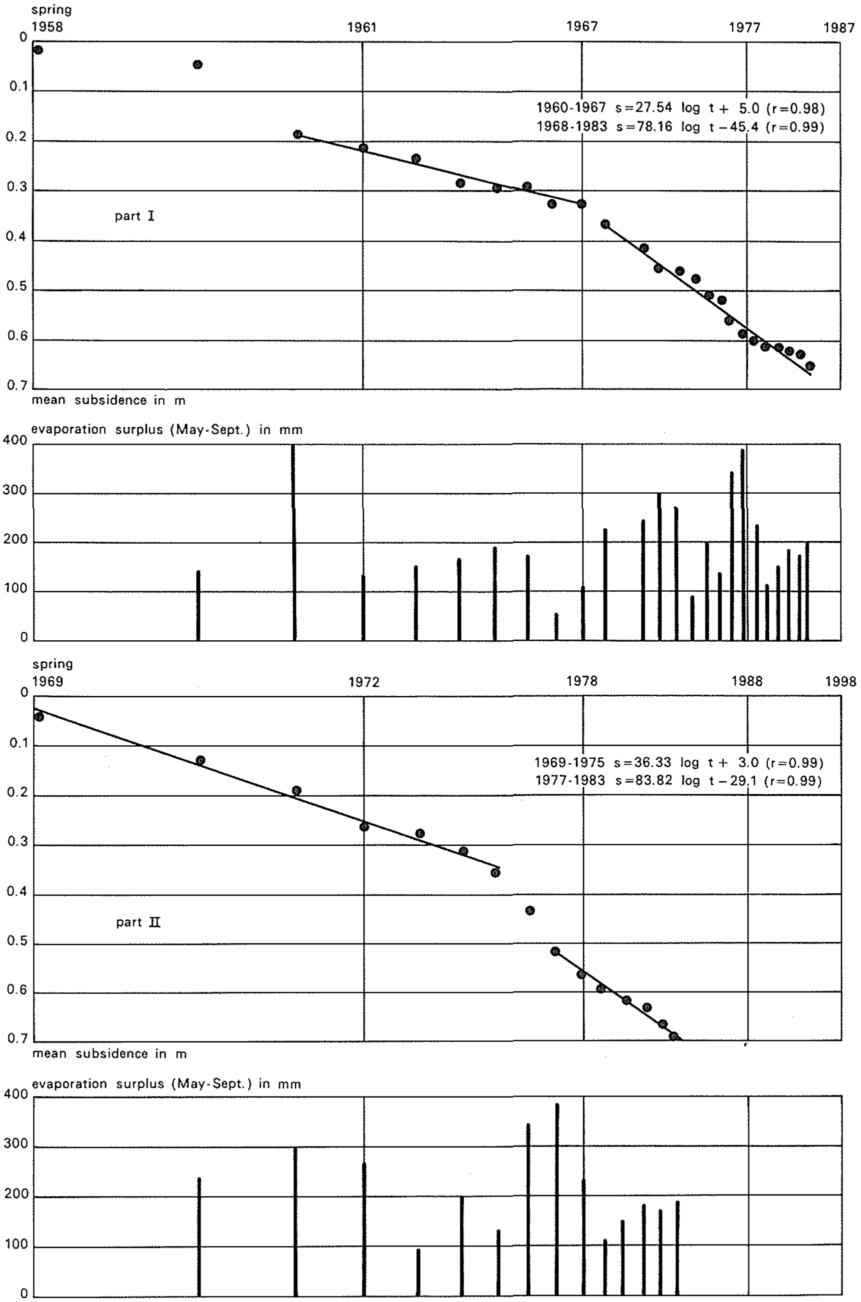

Fig. 5. Mean subsidence in Flevoland in relation to time and the evaporation surplus from May through September in the preceding summer

In part I the high evaporation surplus in the summer of 1959 (405 mm) resulted in a mean subsidence of 0.14 m. The highest value recorded at any site was 0.24 m. Due to the moderate, but gradually improving drainage conditions and the relatively low evaporation surplus in the sixties the subsidence proceeded slower than later on. In the late sixties the drainage system, included the subsurface system, was completed nearly everywhere. This in combination with the relatively high evaporation surplus in the seventies brought about a higher subsidence rate, continuing also in the eighties. It is striking, that the three summers with an evaporation surplus over 300 mm (1970, 1975, 1976) are hardly reflected in the subsidence rate. Evidently, the soil has become so firm in the meantime, that a relatively high rise of the capillary forces has only a minor effect on the shrinkage and therefore on the subsidence rate.

In part II from spring 1969 through spring 1975 the subsidence is very regular. At the first glance, it is striking, that the dry summer of 1970 (surplus 300 mm) is hardly reflected in the subsidence rate. A high value was likely, considering the high value in part I in 1959 and the comparable softness of the soil in part II in 1970. The difference is, apart from the already lower surplus in 1970, that the surplus for July through September 1959 was 215 mm, whereas in 1970 it was only 55 mm. The greater part of part II was still under reed then. Reed developes rather late, so after June only a completely developed vegetation is present, transpiring at its maximum. The summers of 1975 and 1976 have enlarged the subsidence rate, but not up to such extreme values like in 1959 in part I. The strongly drying effect will not have found expression as a result of an already relatively firm soil as compared with the soil in part I in 1959. From spring 1977 through spring 1983 the subsidence rate is again very constant and closely related to the logarithm of time.

In the first period of about ten years after emergence the subsidence proceeds less fast than later on as a result of the initially rather poor, but gradually improving drainage conditions. It is rather well related to the logarithm of time, however, though irregularities occur, due to the influence of the variations in the evaporation surplus on the then still very soft to rather soft soil. After that a very regular and close relation between the subsidence and the logarithm of time is found. So it can be concluded, that also the subsidence resulting from physical ripening of the top soil (desiccation and shrinkage) proceeds with the logarithm of time. This enables to check the subsidence predictions by extrapolation after a couple of years.

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SUBSIDENCE OF PEATLAND CAUSED BY DRAINAGE, EVAPORATION, AND OXIDATION

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Abstract

Drainage, reclamation, and agricultural using of peatlands change their natural conditions, because the layers have originally a high pore volume and high water content. The subsidence of peatland is caused by three main effects, which are (1) compression of peat layers below the groundwater table, if it is lowered, (2) shrinkage of peat layer upon the groundwater table by desiccation, caused by evaporation, (3) oxidation/mineralization of the organic matter in the top layer, especially at arable land, depends on pH-value.

For the predicting of peatland subsidence we have to notice the following points: (1) type, decomposition and density of peat, (2) thickness of peat layer, (3) groundwater lowering, (4) reclamation and using, (5) climate condition.

Two formulae had been given to predict subsidence for predrainage and second drainage.

The shrinkage of the peat top layers depends on the soil moisture tension (pF) and on the climatic and drainage conditions.

For several peatlands in Europe, Northern America and Asia the oxidation rates are correlated with climate dates.

Résumé (in French) see at the end.

Introduction

In Northwestern Germany the peatlands are soils, which man has occupied at last for agricultural use. Every utilization of peatlands (peat cutting, agriculture, horticulture, forestry) requires an adequate water management. Many peatlands of Northwestern Germany lie in lowlands along the bank of rivers near or below the main sea level. These peatland polders, mostly used as grassland, demand pumping stations.

Climate

Northwestern Germany belongs to the humid climate zone of Europe. Perennial winds from west and southwest cause a maritime climate with mild winter and relative cool summer. Table 1 shows the main climate dates.

Table 1. Mean dates of climate of Northwestern Germany

Mean or sum	Air temp. °C	Rel. air humidity %	Rain-fall mm
Year	8,5	84	750
January resp. winter	0,3	90	350
July resp. summer	16,7	78	400

The climate diagram (figure 1) shows the annual course of temperature, rainfall, and potential evaporation for a normal year.

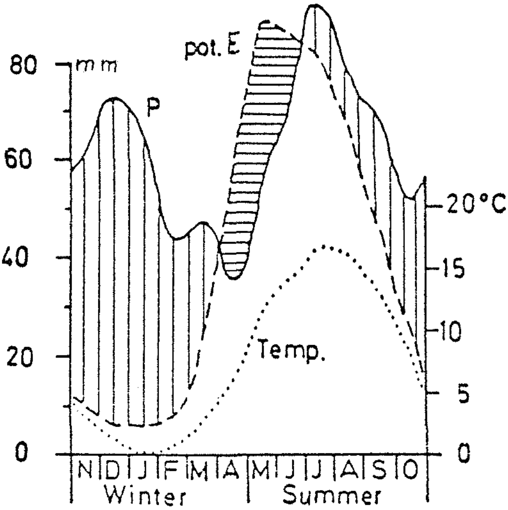

Figure 1.
Climate diagram for peatland pasture in Northwestern Germany

Soil types

A great portion of soils in Northwestern Germany is influenced by high ground-water level or generally by high rainfall. The proportion of peatlands is considerably great (table 2).

Table 2. Soil types in Northwestern Germany

Soil types	Area	
	mill ha	% of total
Gley-, river-, and marsh-soils	1,00	15,6
Peat-soils (peaty, fen, and highbog	0,93	14,5
Sum	1,93	30,1

About 20 - 25 % of these soil types lie in polder areas near the sea coast resp. in the river lowlands.

Peat soil structure

The soil structure is particularly important for the planning of soil meliorative (drainage) measures, because it influences the water storage as well as the air and thermal conditions of the soil. Soil structure is the three-dimensional arrangement of solid soil particles amongst and in relation to each other. The soil volume is subdivided into solid volume and pore volume and the latter is filled to varying degrees by water and air (Fig. 2.9.).

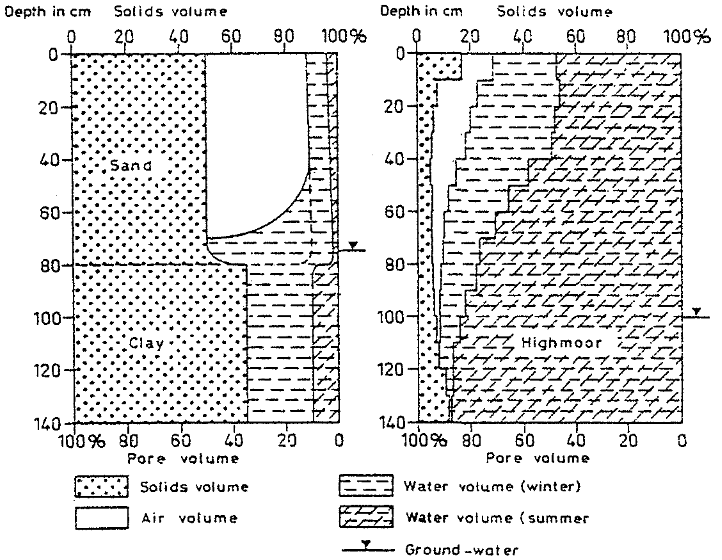

Figure 2. Soil structure of mineral and peat soils.

Peat soils do not have - contrary to mineral soils - a solid supporting structure. All types of peat have an unstable, fibrous, fungilik structure. The high pore space of 97 - 80 Vol.-% is nearly totally filled with water.

Drainage

Every modern utilization of peat soils in the humid, temperate zone needs an adequate drainage. This is attained, if in spring the ground-water level lies about 50 to 60 cm below surface. The experiences of the last decades have shown, that this drainage target is adequate. To calculate the proper drain spacing we need the formula of HOOGHOUDT (van BEERS, 1965).

For peatland drainage we have to respect the consequences of subsidence etc.

Subsidence

Drainage and reclamation of peat mean a disturbance of the natural conditions. This affects the typical properties of peat - high pore volume, high water content - and stops the accumulation of organic matter. Improved drainage and reclamation have three main effects, as:

- subsidence (compression) of the peat layers;
- shrinkage of the top layers;
- oxidation (mineralization) of the organic matter.

Subsidence (compression)

In peat soil the drainage releases physical effects, which never end. The macropore space and the hydraulic conductivity decrease, the bulk density and the subsidence increase.

Figure 3 shows the schematic connection for the whole time of drainage between solid matter content, macropore space, subsidence, and hydraulic conductivity. This phenomenon was found in many peatlands in Western Europe and other parts of the world.

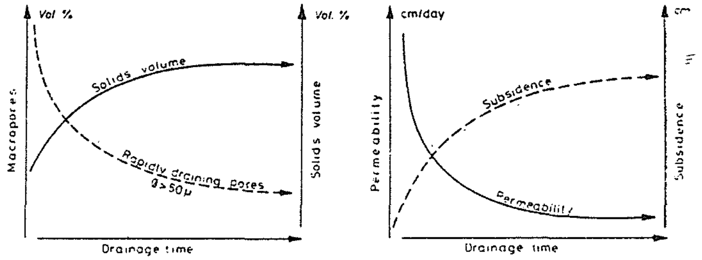

Figure 3. Temporal influence of drainage in peats on soil structure, subsidence and permeability.

If bogland is drained, the structure of the peat is modified. The coarse pores which, up till now, were filled with water, are emptied and later compressed. Consequently the air and water permeability is reduced (Figure 3). The soil dynamics set in motion by drainage, are practically unending and have been known for a long time as peat subsidence.

The surface subsidence to be expected in peat soils, after drainage at a normal drainage depth, may be calculated beforehand according to the empiric subsidence formula evolved by HALLAKOPRI - SEGEBERG (1966).

The subsidence formula runs: $S = a(0.080\ T + 0.066)$; where S = subsidence, T = peat depth (both in m) and a = factor of compacting density, which can be analytically determined via the solids volume. Figure 4 shows the nomogram of the subsidence formula and table 3 the relation between initial drainage, compactness, solid volume and subsidence formula.

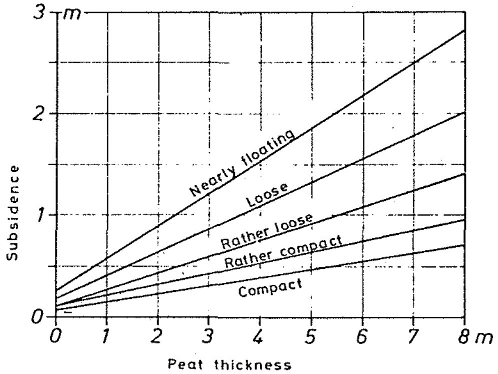

Figure 4. Nomogram of the empirical subsidence formula.

Table 3. Relation between initial drainage, compactness, solids volume and subsidence formula for peat drainage.

Evaluation according to initial drainage (in situ)	Relative compactness	Solids volume %	Factor a in formula	Subsidence formula
None	nearly floating	< 3	4.0	$S=0.32T+0.26$
Very reduced	loose	3 - 5	2.85	$S=0.23T+0.18$
Reduced-moderate	rather loose	5 - 7.5	2.0	$S=0.16T+0.13$
Moderate-good	rather compact	7.5 -12	1.4	$S=0.11T+0.10$
Intensive + long	compact	>12	1.0	$S=0.08T+0.07$

For several peat sites we have compared the measured subsidences with the predicted subsidences. The results of predictions and measurements correspond rather well.

SEGEBERG (1960) has adapted in the subsidence formula the depth to which the ground-water is lowered. This approximation gives the following subsidence formula after SEGEREN (1974)

$$S = \xi D_f D_o^{0} . 707$$

where is S = subsidence (m)

D_o = initial thickness of peat (m)

D_f = final depth of drainage (m)

ξ = coefficient

The coefficient depends on the pore volume (P) as follows

$$\varepsilon = 0.05 + \frac{1}{100 - P}$$

The rates of first surface subsidence after drainage vary between 0.5 m up to more than 3 m. New drainage causes new subsidence. For second or third drainage period the subsidence can be predicted by the following subsidence time formulae (table 4).

Table 4. Subsidence time formulae for second or third drainage periods by ILNICKI (1974).

Relative compactness	Formula*)
loose	$S = 14.3 . t^{0.412}$
rather loose	$S = 7.35. t^{0.476}$
rather compact	$S = 5.14. t^{0.485}$

*Where is S = subsidence in cm and
t = time in year.

The subsidence after second (third) drainage varies between 10 cm and 50 cm for a period of about 30 years.

Peat surface oszillation

In natural peatlands the surface subsides in summer and rises in winter. This phenomenon is caused on the one hand by an evaporation surplus (in summer) and on the other hand by rainfall surplus (in winter)(compare climate diagram in figure 1).

The amplitude is influenced by the evapotranspiration of the different vegetations of fen or highbog. The annual oszillation amplitudes differ between 4 cm (for grassland), 6 - 10 cm (for moss vegetation), and 10 - 20 cm (for reeds); groundwater inflow from a river or lake reduces the oszillation.

Shrinkage

Shrinkage occurs in the topsoil above the phreatic level due to highly negativ soil water pressure heads caused by an evaporation surplus. The degree of shrinkage depends on the thickness of the layers liable to shrinkage and on climatic and drainage conditions. Shrinkage causes crack formation and a considerable increase in hydraulic conductivity, the higher the peats are decomposed, the greater the shrinkage phenomenon will be. The rates of shrinkage vary between 10 mm up to 35 mm (SCHOTHORST, 1982).

Oxidation/mineralization

The oxidation resp. mineralization is a microbiological and chemical process in the top organic layer. It is influenced firstly by climate and type of peat, furthermore by chemical and physical soil conditions, soil moisture and depth of ground-water table, reclamation and cultivation, tillage and plant cover, etc.

On peat grasslands and forests the rate of oxidation is very small and may be compensated by the litter of leaves and/or remains of roots. Therefore the peat oxidation of arable and horticultural lands will be discussed here only.

Peat Consumption in Different Climates

For this analysis we collected many data on the loss of height of peatlands in Northern, Western, Eastern, and Southeastern Europe, in Northern America, and in Asia. We examined these data from soil science reviews. Then we compared these data with the climate conditions.

For this report we returned on the rain factor by LANG (1915) citated by EGGELSMANN (1976).

$$f = \frac{R}{t}$$

where is R = the annual rainfall (mm) and t = the annual mean temperature (°C). LANG has used this factor to explain the main climate influence on the soil types in the different regions of the world.

For our analysis we must take the data of the main climate stations, but we know, that the climate in peat areas is generally influenced by the peat soil itself. There exists many investigations on the peat micro climate. But for this general survey the rain factor by LANG may be sufficient.

For 20 low moor peatlands the annual loss caused by oxidation is graphically plotted in fig. 5 together with the rain factor. A correlation of the logarithm data shows a very high significant correlation (r = 0,729***). The curve shows the relation between the annual loss of height by oxidation and the climate expressed as rain factor. The smaller the rain factor, the higher will be the peat soil oxidation. The listed six regions demonstrate the climate. In semiarid climate the annual loss caused by oxidation increases up to 50 mm/a. Maximum values are 70 mm/a.

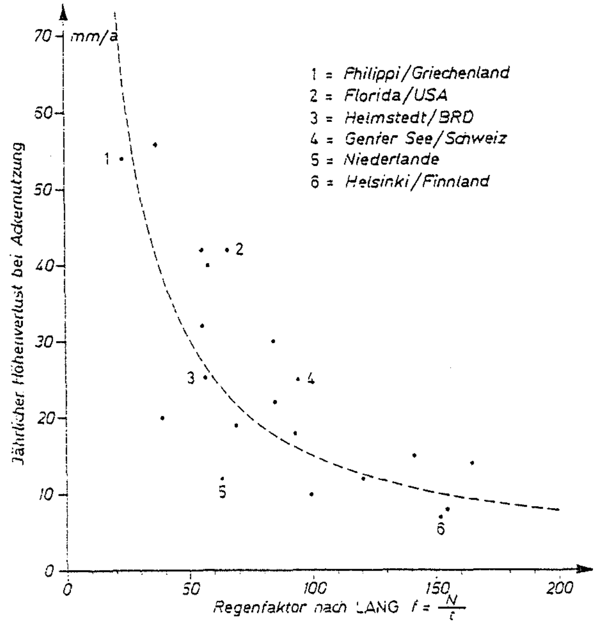

Figure 5. Annual loss of height caused by oxidation of the organic matter in low moor soils (fens).

In fig. 6 the oxidation data of 14 reclaimed high-bog soils are plotted versus the rain factor. High-bogs will be found only in humid climate regions. But fig. 6 shows that first of all the annual loss of height increases with decreasing rain factor until a maximum with a rain factor of about 80 will be reached, then the loss decreases too. This rain factor characterizes as region with a rainfall of 560 to 640 mm and an annual main temperature of 7 to 8 °C.

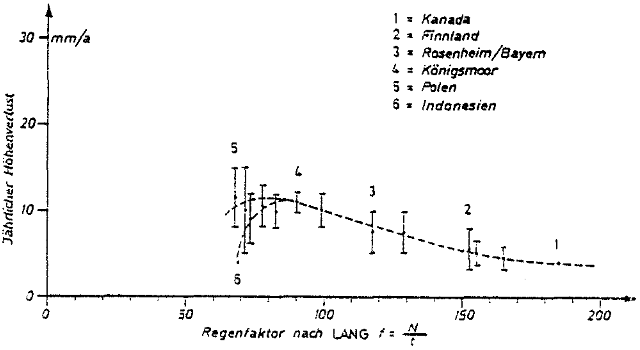

Figure 6. Annual loss of height caused by oxidation of the organic matter in highbog soils.

For peatlands in Indonesia with oligotrophic wood peats the annual peat consumption caused by oxidation is rather small. The short vertical lines in fig. 6 show the fluctuation of the oxidation rate caused by groundwater depth, lime content (pH), kind of crops, and others. For low moor peatlands we have only plotted the means. The variance is larger than in high bog soils.

The rates of height loss caused by oxidation over a period of 30 years vary between 30 cm in (ologotrophic) highbogs and about 60 cm in (eutrophic) fens in humid climate resp. much more than 120 cm in warm climates.

Peatland protection

In the last decade in Northwestern Germany the peatlands without agricultural using got under peat protection, nature conservation or landscape reservation. Many protection areas got a hydrological protection zone.

by the drain spacing formula of HOOGHOUDT (van BEERS, 1969), the results of nearly peatland protection areas show table 5 (by KUNTZE & EGGELSMANN, 1981).

Table 5 Width of hydrological protection zone

Peat type	Width m
deep high bog	30 - 80
shallow high bog above fine sand	120 - 150
shallow low moor above sand	200 - 350
spring-water bog, wooded swamp	> 350

Final remarks

The period of peatland drainage for reclamation ended at about 1965. Only grasslands on peat, which are agriculturally used since several decades, require at some places a better subdrainage or a thin sand cover for an intensive using as pasture. These problems of the agricultural engineering are equivalent to that of nature conservation.

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AFFAISSEMENT DU SOL MARECAGEUX SUITE AU DRAINAGE, A L'EVAPORATION ET A L'OXYDATION

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Résumé

Le drainage, le défrichage et l'exploitation agricole des sols marécageux en changent l'état naturel, car, à l'origine, les couches possèdent un volume élevé de pores et une importante teneur en eau. L'affaissement d'un sol marécageux est le résultat de trois phénomènes principaux qui sont : (1) la compression des couches de tourbe en-dessous de la nappe phréatique si celle-ci baisse, (2) le tassement de couches de tourbe au-dessus de la nappe phréatique faisant suite à une dessiccation, elle-même causée par l'évaporation, (3) l'oxydation ou la minéralisation de substances organiques dans la couche supérieure, notamment dans les terres cultivées, dépend du pH.

Pour prédire l'affaissement d'un sol marécageux, il nous faut noter les points suivants : (1) nature, décomposition et densité de la tourbe, (2) épaisseur de la couche de tourbe, (3) baisse de la nappe phréatique, (4) défrichage et utilisation, (5) conditions climatiques. Il a été donné deux formules permettant de prévoir un affaissement pour un prédrainage et un second drainage.

Le tassement des couches supérieures de tourbe dépend de la tension d'humidité du sol (picofarad) et des conditions climatiques et de drainage.

Pour plusieurs sols marécageux d'Europe, d'Amérique du Nord et d'Asie, on a établi un rapport entre les taux d'oxydation et les données climatiques.

DRAINAGE-INDUCED LAND SUBSIDENCE IN METROPOLITAN NEW ORLEANS, LOUISIANA, U.S.A.

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Abstract

At least two thirds of metropolitan New Orleans is built on reclaimed interdistributary marshes of an abandoned lobe of the Mississippi River Delta. Interdistributary marshes in this region are characterized by accumulations of marsh-grass peat up to five meters in thickness, often interbedded with thin layers of fine silt and clay. Loss-on-ignition analyses reveal that the peat is typically 70 to 80 percent organic matter by dry weight. Below the water table it is approximately 85 percent water by weight.

The marshland was reclaimed by pumped canal drainage, lowering the water table by as much as three meters, and the addition of land fill to compensate for initial subsidence. However, the drainage and land filling process itself causes three types of land subsidence: (1) primary consolidation of the drained peat and underlying clay strata, (2) secondary compression of the peat and underlying clay from the loading of land fill and drained peat, and (3) oxidation of the drained peat.

The low bearing strength of this material requires that buildings be supported by pilings driven at least 12m deep into more competent underlying clay units. Properly spaced pilings stabilize foundations, and, to a degree, retard subsidence directly under buildings, whereas maximum subsidence continues in adjacent areas not protected by pilings. This differential subsidence between buildings and adjacent land results in stress and structural failure of driveways, walkways, and most importantly, the rupture of underground utility pipes.

Differential subsidence of up to one meter has been measured, and the amount of differential subsidence correlates closely with sediment type, the amount of water table lowering, and elapsed time since land reclamation. Thus, it is possible to predict areas of existing and future development that will experience hazardous differential subsidence.

Introduction

The city of New Orleans was founded in 1718 as the southernmost port of the Mississippi River where goods could be transferred to and from river boats serving the vast interior Mississippi Valley. The original city was built entirely on the "natural levee", the ridge of silty sediment that borders each side of the river. At New Orleans the highest part of the natural levee is about 5m above sea level; therefore, the levee provided a relatively dry and firm foundation for building and, when augmented by a low artificial levee system, a measure of protection from flooding. The city was isolated from "mainland" Louisiana by cypress swamps and grassy marshes on the east and west and by Lake Pontchartrain on the north. Thus for nearly two hundred years New Orleans was an "island city" accessible only by the river, the coastal water routes, and the shell roads (frequently washed-out) built on the natural levees. The strategic location of the city, however, more than offset the natural difficulties, and New Orleans grew rapidly. Census figures show that almost from its beginning, New Orleans was one of the largest cities in North America. By 1835, it was virtually in a tie with Philadelphia for second place in population among American cities, and it remains today among the twenty largest metropolitan areas in the United States (Lewis, 1976).

Until the early 1900's the city was restricted to the relatively narrow levees of the Mississippi. This situation changed abruptly when inventor-engineer Baldwin Wood designed a heavy-duty pump that made it possible to quickly raise huge volumes of water a short vertical distance. Drainage canals were dredged through the cut-over cypress swamps north of the city, and Mr. Wood's pumps were used to drain the land. Artificial levees were constructed to protect the newly drained land from flooding. By 1920 developers were building on this land, much of it near or slightly below sea level. It was soon discovered that conventional houses could be built successfully in the drained swamp lands without the use of pile-supported foundations. Construction continued until shortly after World War II, by which time most of the old cypress swamp had been reclaimed and developed.

The remaining undeveloped "land" in Orleans and adjacent Jefferson Parishes was the brackish-water marsh along the southern shore of Lake Pontchartrain. This marshland was drained by the same type of canal-and-pump system which had been used earlier in the cypress swamp. However, the area is underlain by as much as 5m of marsh-grass peat, which provedes a poor substrate for construction. Subsidence of the land surface became a major problem in the newly drained areas because the underlying organic-rich sediment was easily compressed. Today large parts of the New Orleans metropolitan area must still cope with the damage caused by sinking land.

Geologic Setting

From the beginning, the way of life in New Orleans was greatly influenced by the underlying geology. The location of early settlements, the style of buildings, the routes of streets and highways, the drainage systems, and even the patterns of ethnic populations in older parts of the city, all are reflections of the geology of the Mississippi River Delta.

The delta is constructed of billions of tons of mud and sand that were eroded from the interior of our continent, transported southward by the Mississippi River, and dumped where the river entered the sea. The flat, low-lying land area built seaward by the deltaic accumulation is a complex of stream channels, levees, swamps, marshes, and lakes, the whole

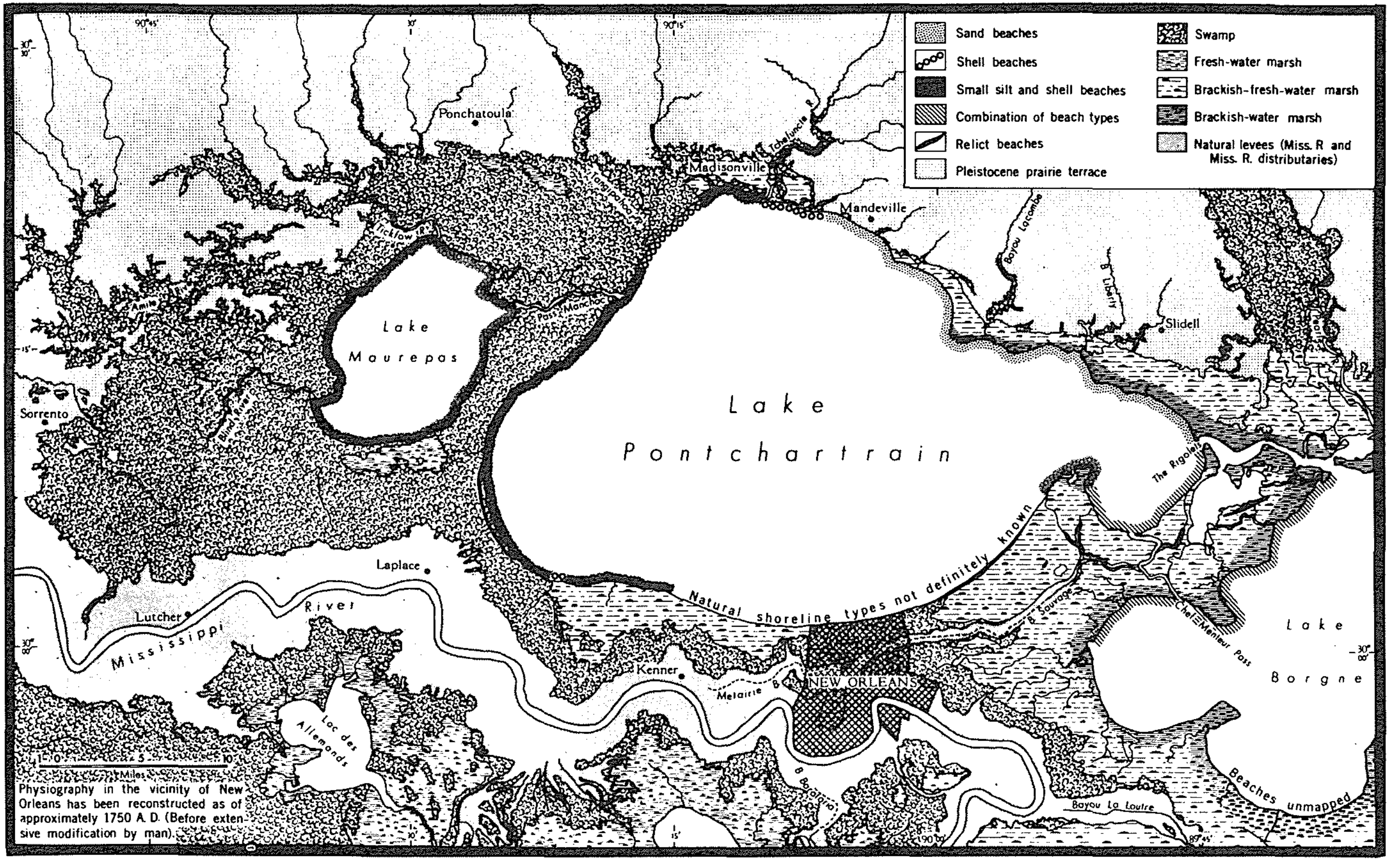

FIG. 1 Sedimentary depositional environments and landforms in metropolitan New Orleans. Circa 1750 (from Saucier, 1963).

of which is called the "delta plein". Figure 1 shows a portion of the Mississippi River Delta Plain in the vicinity of New Orleans. The Mississippi River Delta region of southern Louisiana is quite young, geologically speaking, and the deltaic sediments are still soft and unconsolidated.

Recent Geologic History of New Orleans

During the Wisconsin Glacial Stage, the area that is now southern Louisiana stood perhaps a hundred meters above sea level. Then, about 10,000 to 15,000 years ago, the eustatic rise in sea level began to affect the region. Gulf water flooded the New Orleans area about 5,000 to 6,000 years ago, and the Mississippi River began to build its delta in the area southeast of Lafayette, La. In the last few thousand years the river has changed course several times, and deltas have accumulated at various sites from Franklin, La., to south of Biloxi, Miss. The flat low-lying land area built seaward by the deltaic accumulation is a complex of stream channels, natural levees, swamps, marshes, and lakes--the delta plain (Figure 2).

5,000-6,000 years ago, before the beginning of extensive deltaic sedimentation in the vicinity of New Orleans, a series of northeast-southwest-trending sand deposits extended from the Mississippi coast well into the New Orleans metropolitan area. These are barrier-island, bar and shoal sands that were drifted westward by longshore currents. Saucier (1963) called these sands the "Pine Island beach trend." Although this sand trend was buried by younger Mississippi Delta sediments, it is now in many places only a few meters below the surface, and thus strongly influences subsurface engineering properties. Figure 3 is a map showing the location of these buried barrier islands, along with the Pleistocene land surface contours and several more recent geologic features.

As the river continued to deposit sediments at its mouth, land masses grew progressively farther seaward. But the great weight of the deltaic sediment also caused sinking of Earth's crust beneath the delta complex. When the river changed course upstream and abandoned a deltaic lobe, that part of the delta plain continued to subside, becoming progressively more inundated by the Gulf.

Deposition of the St. Bernard lobe of the Mississippi Delta began in the New Orleans area about 4,700 to 4,500 years ago. It is important to understand the stages of deposition and resulting sediment types, because those sediments now make up the land surface and shallow subsurface of New Orleans.

An upstream diversion--similar to the one presently affecting the Mississippi as the Atchafalaya River enlarges--initiated each of the pre-modern Mississippi River courses. Stream capture was a gradual process involving increasing flow through a diversional arm, which offered a shorter route to the Gulf. After capture was effected, each new course lengthened seaward by building a shallow-water delta and extending it gulfward. The onshore part of the delta surface consists of stream channels, called distributaries, which are flanked by low natural levees. Between the distributaries are troughs

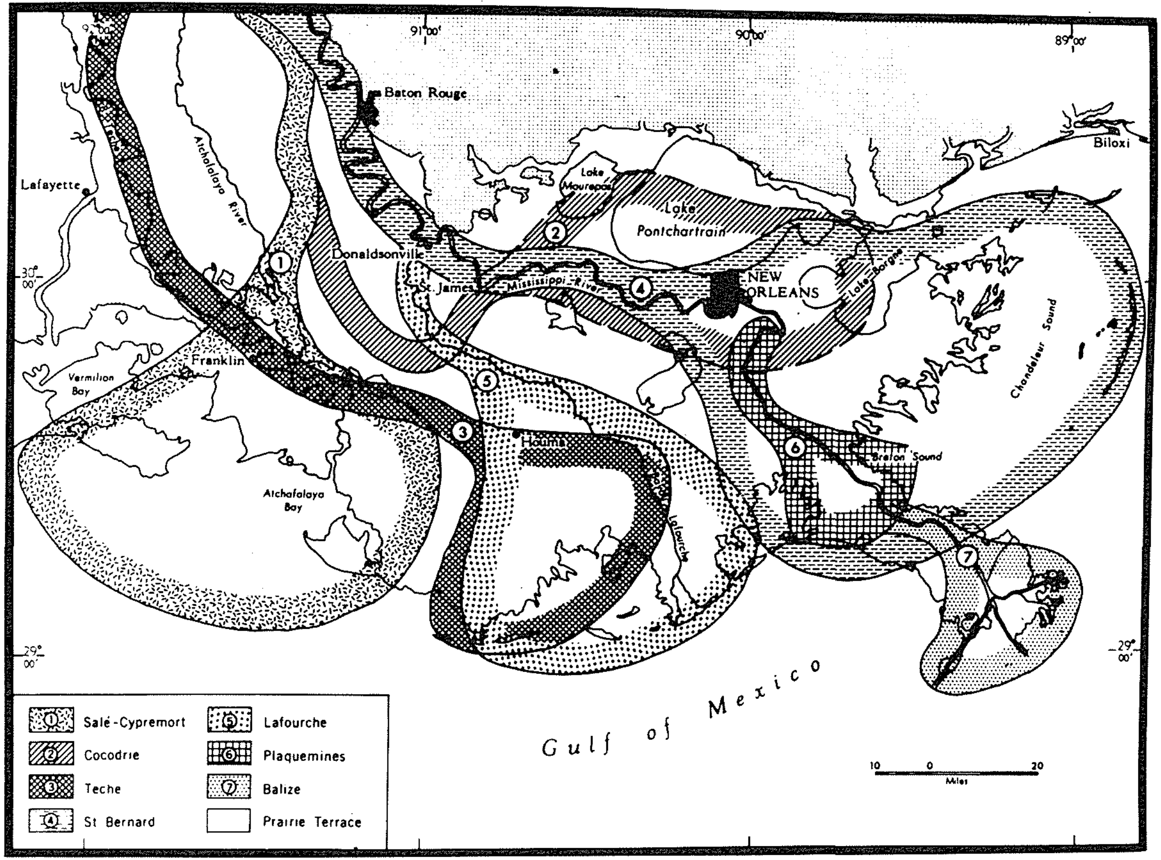

FIG. 2 Multiple Deltas of the Mississippi Deltaic Plain
(from Kolb and van Lopik, 1958)

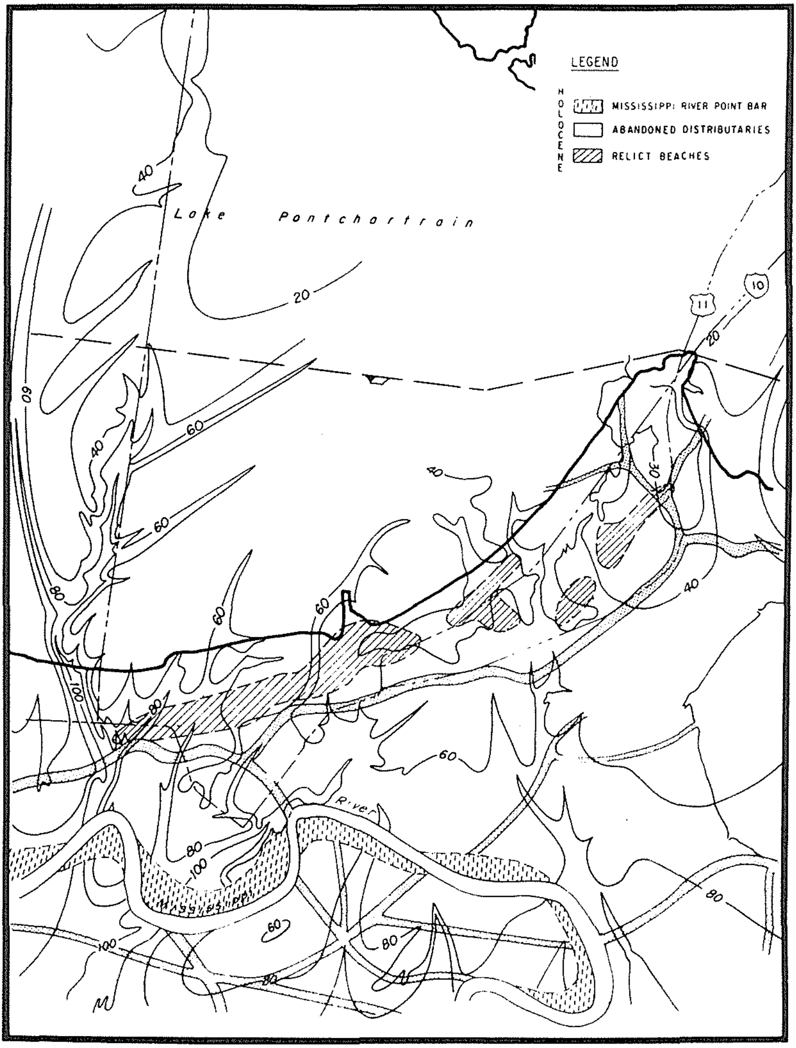

FIG 3. Major geologic features of metropolitan New Orleans. Numbered contour lines represent the depth below mean sea level of the buried late Pleistocene land surface. These contours also show the land surface as it was 5,000-6,000 years ago. Other features shown are: the buried islands discussed in text; abandoned St. Bernard Delta distributary channel deposits of the modern Mississippi River. The dashed line in Lake Pontchartrain is a major east-west trending fault that was active during late Pleistocene time (adapted from Kolb, Smith and Silva, 1975).

that hold near-sealevel marshes and bodies of shallow water. Channels of principal distributaries extend across the gently sloping offshore surface of the delta to the inner margin of the steeper delta front, where the distributary-mouth bars are situated. The offshore channels are bordered by submarine levees, which rise slightly above the offshore extensions of the inter-distributary troughs.

As it lengthens its course, the river occupies a succession of distributaries, each of which is favorably aligned to receive increasing flow from upstream. The favored distributary gradually widens and deepens to become the main stream. Its natural levees increase in height and width, and marshland develops in the troughs adjacent to the distributary. Levees along the main channel are built largely during flood stage. Crevasses create abnormally wide sections of the levee and of adjacent mudflats and marshes, and some of the crevasses continue to remain open and serve as minor distributaries while the levees increase in height. Crevasses also occur along the main stream during flood stage and permit tongues of sediment to extend into the swamps and marshes for considerable distances beuond the normal toe of the levee.

Distributaries with less-favorable alignment are abandoned during the course-lengthening process, and their channels are filled with muddy sediment. The marshlands below New Orleans are beined with abandoned distributaries associated with development of the Mississippi's present course.

The continual migration of various environments of deposition produces a highly complex body of sediment. The sediment type under the delta plain varies from place to place and from depth to depth. Most of the sediments are fine grained throughout the region, reflecting the type of sediment load transported by the Mississippi while the deltaic plain was being built. About 75% of the present-day river load is silt and clay; the remainder is fine sand. Sands are deposited in bars at the mouths of distributaries and in thin sheets spread by marine currents at the delta front. Natural-levee deposits are wedges of silty clay that reach a maximum thickness of 9m at the margins of main channels and thin away from the channels. Organic-rich muds that were deposited on mudflats and in marshes and swamps fringe the natural levees.

Accumulations of peat and organic muck are widespread in several sections of the coastal Louisisna lowlands. Data from many borings provide details concerning local distribution of the deposits in the deltaic plain. The peats range in thickness from a few centimeters to more than 6m, depending on the duration of organic accumulation and the amount of local subsidence. Peat is largely confined to intersistributary troughs of abandoned deltas where continued subsidence allowed marshes to flourish for long periods. Generally, the thickest peat deposits are in the levee-flank depressions along the active and abandoned minor river channels.

The diagrams of Figure 4 show stages in the development of a typical peat deposit; they indicate the changing character of vegetation during levee enlargement and after abandonment of the distributaries.

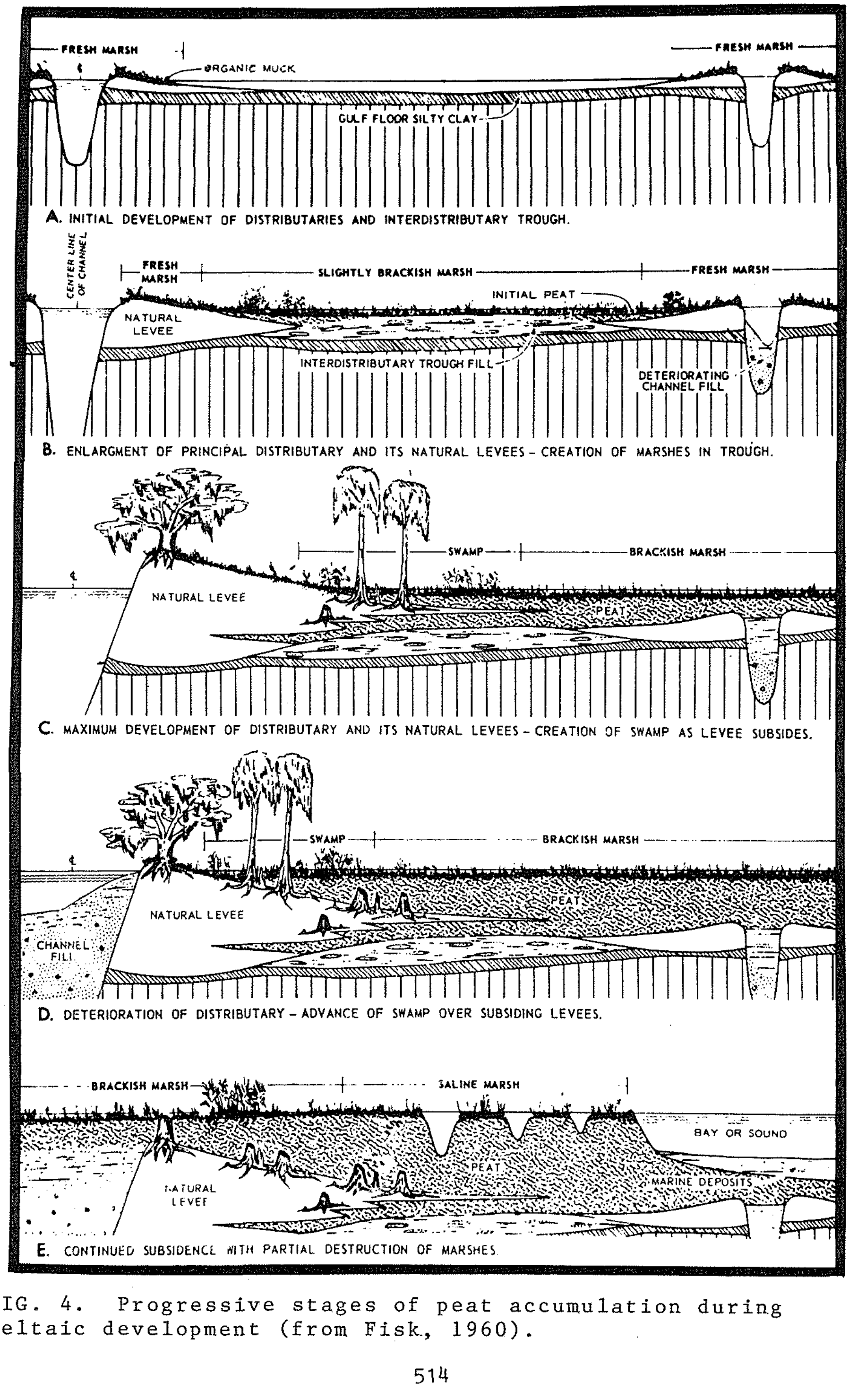

FIG. 4. Progressive stages of peat accumulation during deltaic development (from Fisk, 1960).

Fresh-water plants are first to appear on mudflats in the delta (Figure 4a). Peat begins to form from the remains of cattails, sedges, and grasses in slightly brackish marshes no more than 0.5m above sea level (O'Neil, 1949). Marshes develop over broad areas within the interdistributary troughs during enlargement of the levees (Figure 4b). In the central part of the trough, in areas removed from river sedimentation, peat may develop entirely from marsh vegetation as the trough subsides. Along the margins of a subsiding trough, the organic accumulations reflect a progressive change in vegetation accompanying levee enlargement, from fresh-water marshes through cypress-gum swamps to brackish and saline marshes. Swamps developing in levee-flank depressions shift toward the center of an interdistributary trough while a distributary enlarges and its levees widen (Figure 4c). After a distributary is abandoned and river sedimentation ceases, continued subsidence of the levees and adjacent trough results in a progressive change from swamps to brackish-marine and saline marshes (Figure 4e). Finally, enlargement of the water bodies obliterates the marshes, and peat accumulation ceases.

Wave action and flooding associated with the enlargement of the coastal water bodies may destroy peat accumulations, or it may bury them with marine silts and sands. For example, peat underlies a thin cover of sandy marine sediment in the northern part of Chandeleur Sound (Kolb, 1958). Some of the sands derived from destruction of the deltaic plain by marine processes are swept to the gulf shore where they are incorporated in delta-martin islands. Typical of these is Grand Isle southwest of New Orleans, where sand more than 10m thick rests upon peat-bearing marsh deposits (Fisk, 1955).

Land Subsidence in Metropolitan New Orleans

Subsidence, the relative lowering of the land surface with respect to sea level, is a natural consequence of deltaic sedimentation in the New Orleans area. Surface and subsurface drainage and development in the city also have caused the surface to subside at an increasing rate. The amount and rate of sinking relate to the complex geology of the delta.

Saucier (1963) calculated the average rate of general subsidence in the New Orleans area to have been 12cm per century for the past 4,400 years. This figure is based on radiocarbon dates of peat deposits and does not include the estimated rate of sea-level rise during this period. On a smaller scale, the process is acting on individual landforms at different rates. For example, natural levees and barrier sands, due to their higher bulk density, may actually subside faster than surrounding clay and organic sediments.

According to Terzaghi (1943) land subsidence occurs as a result of three principal causes (see also ASTM, 1965):

(1) Primary consolidation is the reduction in volume of a soil mass caused by the application of a sustained load to the mass and due principally to a squeezing out of water from the void spaces of the mass.

(2) *Secondary compression* is the reduction in volume of a soil mass caused by the application of a sustained load to the mass and due to the adjustment of the internal structure of the soil mass after the water is squeezed out.

(3) Oxidation of organic matter results in the reduction in volume of a soil mass as chemical reactions occur which cause the organic matter to decompose into its mineral constituents.

When the level of the groundwater (water table) is lowered, the material above the new water table is no longer buoyed up by the subsurface waters. Therfore, an increased load is placed upon all material below the new water-table elevation. Deep strata, both organic and inorganic, then undergo primary consolidation and secondary compression over a period of years. Additional compaction and subsidence are caused by the interaction of oxidation and secondary compression in the material above the new water table. Whether the volume change is due to primary consolidation, secondary compression, or oxidation of organic matter, the total amount of subsidence is directly dependent upon the level to which the water table is lowered by drainage.

Relationship of Subsidence to Sediment Type

When a part of a delta is drained for urban development, such as in metropolitan New Orleans, *subsidence may be generally* accelerated, and different rates among the deltaic sediment types are very apparent:

(1) The natural levee-crevasse silts and sands are affected the least. As these deposits formed the high ground (up to 5m above sea level), most were not completely water saturated at the time of development. Further, as these coarser sediments have a grain-support internal structure, they are only slightly affected by dewatering of pore spaces. The same is true of the barrier-island sands.

(2) Backswamp and interdistributary-trough clay deposits, which underlie much of the cypress swamp (Fig. 4) in the New Orleans area are subject to shrinkage upon drying, as the internal structure of these clays is partly water-supported. However, the low permeability of these clays usually prevents them from drying to more than a few feet below their exposed surface, and, therefore, subsidence is often minimal. Where the organic matter in these clays is more abundant their subsidence potential is increased because the organic matter increases permeability and allows deeper drying. Buried logs and stumps in these deposits may provide pathways for moisture loss, and they decompose when exposed to air, thus causing irregular hummocky subsidence.

(3) Peat deposits in the marsh area (Fig. 4) are highly permeable and have by far the greatest potential for subsidence when drained. Particularly large amounts of subsidence occur when the upper peat is left exposed to the atmosphere and shrinkage occurs. Desiccation (drying) of the highly organic soils results in extremely large capillary forces acting to

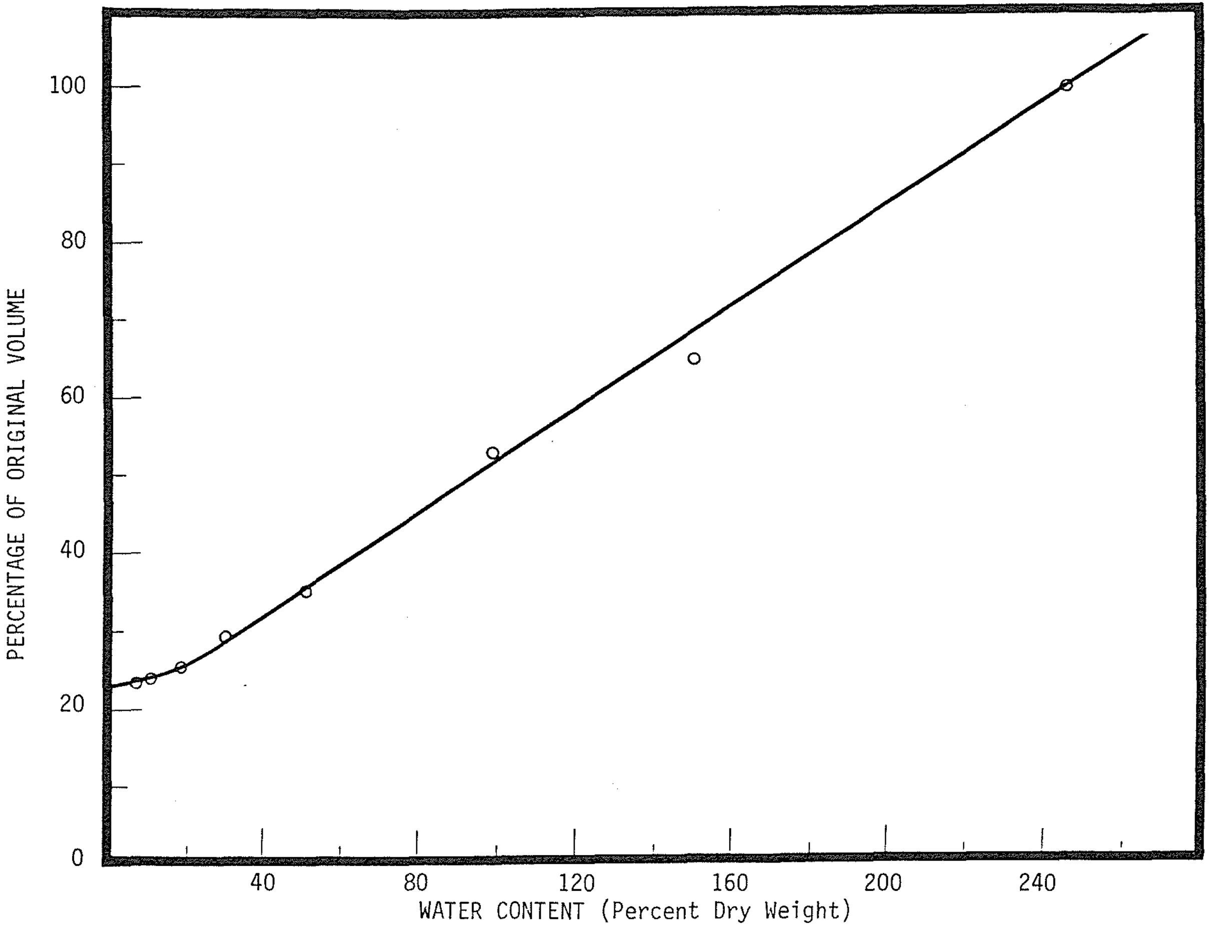

FIG. 5. Shrinkage test results.

compress the upper layers of soil. The compressive forces are much greater than those imposed by the overburden so that the soils are overconsolidated to the point of forming a stiff upper crust. The desiccation also allows oxidation of the organic matter to occur. Although decomposition contributes to the volume change of the upper organic layers, the shrinkage caused by the large capillary forces associated with desiccation appears to be the predominant factor. This is illustrated by the shrinkage test results presented in Figure 5. Shrinkage tests were performed on organic soils by determining the volume of large undisturbed samples with vernier calipers as the samples were allowed to air dry. The samples were weighed in order to determine the water content corresponding to each volume determination. Representative results are presented in Figure 5 as percentage of original vilume versus water content. The original volume was taken as that volume at the natural water content. The results generally show a shrinkage to an ultimate volume of 25 to 30 percent of that volume corresponding to a natural water content of 250 percent. The shrinkage would be even greater if the natural water content were greater than 250 percent. These tests were performed over approximately a 2 week period so that decomposition was not a major contributor; the organic content before and after the test was essentially the same.

General History of Subsidence Problems in New Orleans

Prior to the mid-1950's, most of the construction in metropolitan New Orleans had been on the natural levees of the present Mississippi River and its former distributary. A few subdivisions had also spread into the drained cypress swamp. Most residential construction was on raised-floor foundations, supported by masonry pillars. Sometimes wooden pilings were driven to support the foundation pillars, but more often they were not. Most of these homes are still standing, although irregular subsidence requires periodic foundation levelling in some neighborhoods.

An unfortunate coincidence was the widespread change to the concrete-slab foundation system by residential contractors at the time of urban development of the marshlands in the New Orleans area. Some of the early construction of the reclaimed marshland proved disastrous. The soft spongy peat failed to support heavy concrete slabs, which simply sank into it, occasionally tilting and breaking in the process. It was soon discovered, however, that if enough wooden pilings were driven 10 to 15m through the peat into the clay below, the friction on these pilings would support the slab. In February of 1979, some 25 years after the initial development of marshlands, the Jefferson Parish Council passed an ordinance requiring residential contractors to use pilings in the thick-peat areas of the Parish.

As had been the case in the cypress swamps reclaimed earlier, the drained land surface of the marsh is so low and so hummocky that it is necessary to add up to 1m of fill to level and ele-

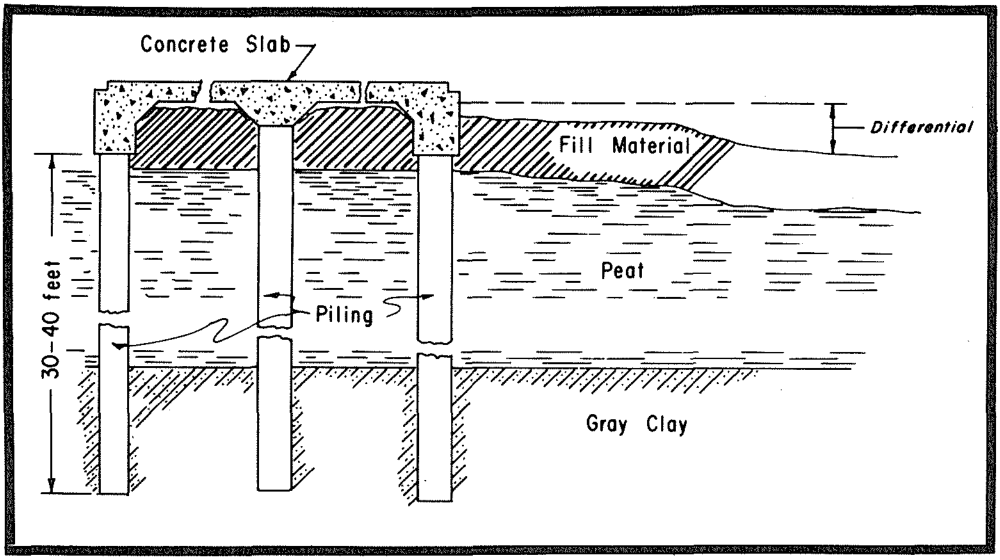

FIG.6. Section of slab-on-piling construction showing differential subsidence.

vate building sites. This is usually done on a lot-by-lot basis, and a variety of fill materials have been used, ranging from broken concrete and asphalt to topsoil. Sand dredged from the Mississippi River or other nearby river channels is the prevalent fill material for residential sites at the present. Figure 6 is a cross section of the slab-and-piling foundation, which has been used for residential construction in the reclaimed marshland since the late 1950's.

Differential Subsidence

Probably the greatest single problem has not been the general areal subsidence but the difference in subsidence between houses on pile foundations and the surrounding ground surface. When houses or buildings are constructed using the slab-on-pilings technique, the foundation is stabilized, but the area surrounding the the building continues to subside, thus producing differential subsidence. Many homeowners fill their yards with 10 to 20 cubic meters of soil each year to compensate for this differential subsidence.

Figure 6 shows a representative cross section of a house foundation on piles and the surrounding area after differential subsidence has occured. A gap may occur under the house slab, but generally the material immediately surrounding the piles adheres to the piles so that the gap beneath the slab is much less than the total differential subsidence. Because the material near the pile foundation is actually supported to a certain extent by piles, the effect is usually one of greater surface subsidence farther from the house. The most important factor is the magnitude of the differential subsidence between the house slab and the surrounding area.

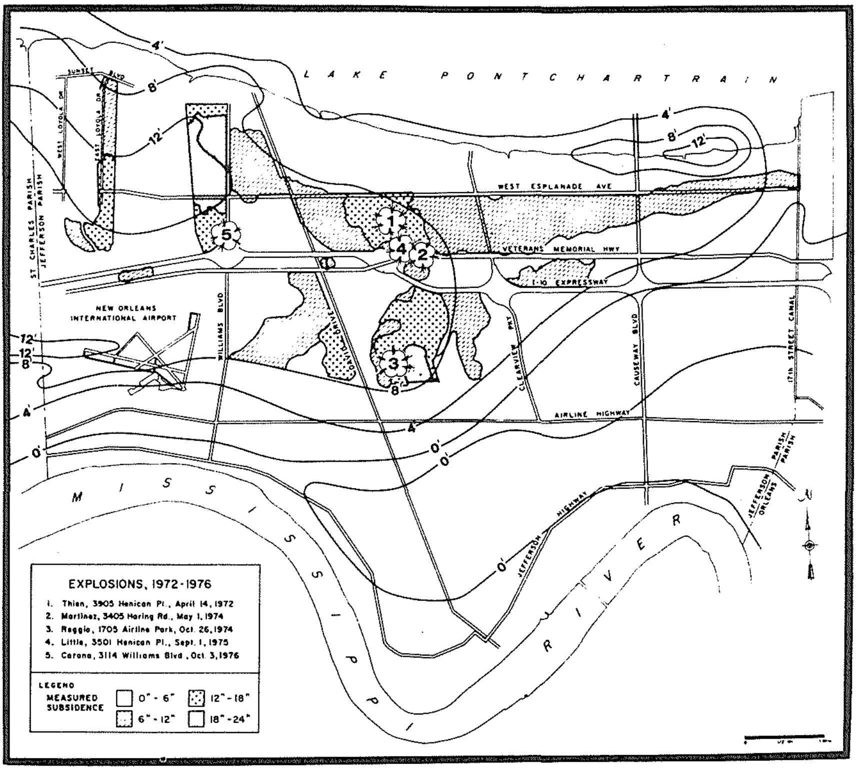

FIG. 7 Measured differential subsidence in Jefferson, Parish, La. related to peat thickness. Blank area in western part of map is undeveloped.

Hazards and Damage Due to Differential Subsidence

Major effects of subsidence have been widespread damage to sewer-, water-, and natural-gas lines, and to streets, driveways, and sidewalks, as well as to structures. Recent case studies have revealed tilting of houses over filled canals, negative skin friction on poles, cracked slabs, and other types of structural distress. The general difficulties are too numerous and the complete ramification of subsidence damage is too lengthy to present in this paper; only the worst hazard caused by differential subsidence will be discussed.

As troublesome as subsidence-caused maintenance problems are, the greatest hazard in the marshland peat area is from natural-gas explosions. Gas and other utility lines are buried in the peat. The stress created by differential subsidence is sometimes enough to rupture gas lines, releasing gas into the highly permeable drained peat. If the fill layer is less permeable than the peat, the gas may migrate some distance, eventually accumulating under a concrete slab foundation. Since 1972, five homes have been destroued by natural-gas explosions. Figure 7 is a map published by Louisiana Gas Services Company showing measured differential subsidence rates and recent explosion sites in Jefferson Parish. Lines indicating peat thickness are superimposed for reference. All the explosions are believed to have been caused by subsidence-related gas line ruptures.

Kenner, Louisiana- A Case History

Kenner, Louisiana is a thick peat area located in the western part of the metropolitan New Orleans area.
The history of subsidence in Kenner is representative of what has occured in much of the drained and developed marshland. Examination of this subsidence history should provide a guideline for establishing the proper waiting time between drainage and construction in order to mitigate subsidence-related damage. The following case history of land subsidence was reported by Traughber, Snowden and Simmons (1978).

Early condition-Prior to 1924

Orginally the areas of lower elevation in Kenner were marsh and swamp, similar to those now existing in St. Charles Parish to the west. Inasmuch as the level of Lake Pontchartrain adjacent to the Kenner area is nearly 30cm above mean sea level (MSL), it can be concluded that the original marshland was at this same elevation. The upper layers were generally very light and, in many cases, tended to float. The dry-unit weight of this material could be as little as 0.2g/cc.
Drainage of this type of land results in large amounts of immediate subsidence as the unstable upper material compresses.

Beginning Development-1924 to 1949

Initial drainage pumping started in the mid 1920's, with the old New Orleans-Hammond Highway (State Highway 33) acting as a protective embankment along the shore of Lake Pontchartrain.
By 1946 the crest of this load embankment had been reduced from an elevation of + 1.5m MSL to approximately + 1m MSL due to wash or subsidence. The four pumping stations in Jefferson Parish were poorly maintained and one was taken out of operation in 1932 because of a break in the foundation. The pumping primarily benefited those areas of Kenner closer to the river, and the northern areas remained swampy throughout the period of 1924 to 1949.

Development after 1949

On September 19, 1947, a hurricane struck the Lake Pontchartrain area, causing sustained flooding in the lower-lying areas of Jefferson Parish. The old New Orleans-Hammond Highway embankment was breached and overtopped. The flood damage which occured was the impetus for construction of a new levee-protection system along the lakeshore and the Jefferson Parish-St. Charles Parish line. Improvements in the pumping capacity of the drainage system also were implemented. The levee and drainage improvements began in earnest about 1949 and were completed by 1953. This provided the first opportunity for major housing construction in the northern section of Kenner. The first residential development in northern Kenner, begun in 1953-54, had unpaved shell roads. The development continued and accelerated with the construction of major thoroughfares such as Interstate Highway 10 and Veterans Highway. Improvements to existing streets and construction of new ones brought the development to its present stage.

Subsidence 1924 to 1935

As previously stated, the elevation of the original marshland in 1924 was + 0.3m MSL. A 1935 survey shows that subsidence had lowered the elevation of the area near the lakeshore, generally north of present-day 42nd Street, between sea level and + 0.3m MSL (United States Geological Survey, 1938). The 1935 survey shows the subsidence had caused an interior basin to form. This low-lying area- generally between sea level and - 0.3 MSL, acted as a temporary holding reservoir during periods of heavy rain. The area to the south of the interior basin increased in elevation toward the river.

Subsidence 1935 to 1949

In 1949 the low lands in Jefferson Parish were still swampy and supported heavy growths of wild cane, sawgrass, and other tropical and semi-tropical vegetation. Pumping was still beneficial, primarily only to those areas of Kenner nearest the river. The pumping-stations were generally poorly maintained and only 5 of the original 8 pumps were still operating. The New Orleans-Hammond Highway embankment had deteriorated or subsided so that it was inadequate for protection against even normal high tides in Lake Pontchartrain. Some measure of protection against flooding was provided by maximum subsidence areas, which served as catch basins for rain water that slowly drained by pumping. These conditions and the fact that the area near the lake was still at an elevation between sea level and + 0.3m MSL (U.S. Army, Corps of Engineers, 1948) indicate that no substantial areal subsidence occured between the years of 1935 and 1949.

Subsidence after 1949

A survey made in 1970 by the Corps of Engineers shows the same basin-like topography that existed in 1935. The interior area was on the order of 0.3 to 0.6m lower than the strip of high ground near the lake. However, comparison of the 1970 survey shows a general overall subsidence of approximately 1.2m in the part of Kenner north of Interstate 10 highway. Since there apparently was no appreciable lowering of the water table or subsidence during the period 1935-49, it has to be concluded that approximately 1.2m of subsidence occurred in that area during the 21-year period between 1949 and 1970. This has been confirmed by field measurements of areal subsidence near houses that were built at different times.

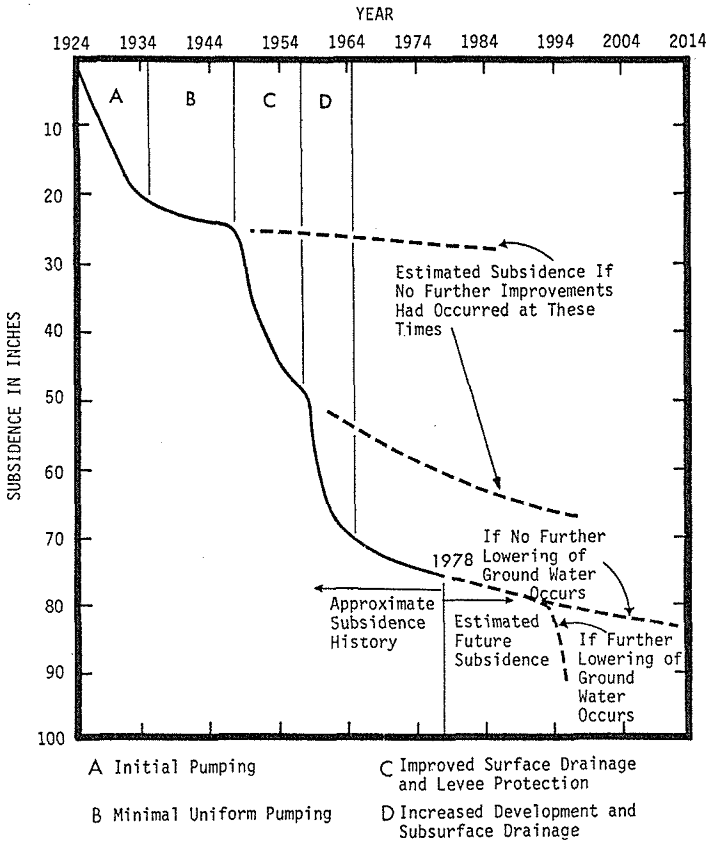

FIG. 8. Approximate subsidence history and estimated future subsidence for Kenner, Louisiana, north of Interstate 10. Normalized for peat thickness of 2.5m.

Because of better drainage, the areas along the southernmost part of the interior basin experienced even more subsidence. Comparison of data from 1935 to 1970 indicates that there was approximately 1.5m of subsidence between those dates along the southern boundary of the interior basin. This is in an area with thick layers of peat (4.6-5.5m) overlain by thin layers of clay. There are small local areas with subsidence probably as much as 2m near canals where the drainage drawdown was greatest.

Time-Subsidence Curve - 1924 to 1978

Field measurements were taken to determine the differential subsidence which has occured at residences built at various times since about 1949. These measurements and information on subsidence for the period 1924-49 were used to determine the approximate subsidence history for the area of Kenner north of Interstate 10 Highway (Fig. 8). The curve represents the subsidence history for a location with an average peat thickness of 2.5m.

The time period from 1924 to 1935 (Fig. 8) represents the initial drainage of the area after construction of the Hammond Highway embankment. During this time the water table was lowered. At the end of this period the water table eas still approximately at the ground surface, but the elevation of the elevation of the ground surface was 0.5 to 0.7m lower than it was in 1924. The period between 1935 and 1949 was a time of minimum and relatively static drainage levels (Fig. 8). During this period the embankment and pumps were often in a state of disrepair, with several completely out of operation. Little subsidence occurred during this time period, and if no further improvements in drainage or levee protection had occurred, the settling would have followed a curve something like the upper dashed line in Figure 8.

The new levee and improved drainage system caused the subsidence rate to increase in 1949. Although some remedial measures were taken to improve the flood protection system after the hurricane of 1947, it was not until 1949 or 1950 that significant construction began on the levee system. This levee was essentially finished by 1952 or 1953, and the first residential development north of Interstate Highway 10 was occupied in October 1954. The real-estate developments at that time had shell roads and side ditches for surface drainage.

Increased subsidence rates occurred during the period of 1949-59 (Fig. 8). As the new pumps and the new levee system improved the drainage, there was substantial lowering of the water table. The rate of subsidence slowed toward the end of the 1950's as the water table again held fairly constant during a period of slower development. The subsidence rate would have continued to decrease as estimated by the middle dashed line in Figure 8 if no further improvements in drainage had been made.

In 1959 or 1960 construction in the area north of Interstate Highway 10 accelerated with a dramatic increase in the number of paved streets. The roof areas and paved areas greatly reduced the open ground available for absorption of rainwater so that the volume of surface runoff increased. This caused both general lowering of the water table and increased demands on the pumping system. The majority of the subsidence due to this increased development probably occured between 1959 and the mid 1960's. Since the mid 1960's the subsidence rate has slowed, and it is now between 1.2 and 2.5cm per year. If no further changes in the water-table occurs, the rate of subsidence should continue to decrease. For this condition, the estimated future subsidence is shown by the lower dashed line extending beyond the year 1978 in Figure 8. However, if there is further lowering of the water table, there will be another period of rapid subsidence, as indicated by the steeper dashed-line curve. Therefore, any new drainage projects in the area must be carefully designed to avoid further lowering of the water table.

Summary

Now that the relationship between drainage and land subsidence is relatively well understood in the region, it is possible to predict areas that have the potential for future hazardous subsidence. Present drainage systems should be modified and future systems designed to keep subsidence-prone sediments and soils as wet as possible. Water table levels should be kept as high as possible. Thorough geological and geotechnical surveys should be done prior to further drainage projects within the Mississippi River delta plain.

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PRESERVATION OF PEAT SOIL BY CULTIVATION OF PERENNIAL HERBAGE CROPS.

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Abstract

On a peat-soil of the reclaimed Hula Valley swamp containing 30-40% organic matter, 1.5% of N, C/N ratio of 10 and pH = 6-6.5 three varieties of Guinea grass, alfalfa and Rhodes grass were grown. The aim of the research was to preserve the peat-soil from the prevailing aerobic oxidation, wind and water erosion and minimize the typical nitrate accumulation, which causes polution of the water sources drained to the Kinneret Lake (Sea of Galilee). Preservation of the peat-soil may lead to minimizing the severe subsidence of the peat-soil surface: 9-10 cm/year during the last 25 years. The investigation was conducted during the last five years in several sites of the Valley and results showed: Guinea grass, Rhodes grass and alfalfa yielded an average dry matter content of 43-44, 30-35 and 25-30 t/ha, per year respectively. A significant decrease of peat-soil erosion and nitrate accumulation was recorded under the above mentioned perennial crops.

Introduction

The Hula Valley is located in the north of Israel, at the lower end of a catchment area roughly ten times its own size. Due to a lava-blocked outlet, the rate of water discharge from the valley is considerably smaller than the rate of inflow through numerous streams. This situation resulted in severe flooding and drainage problems, with the formation of extensive swamps. In 1958 the Hula swamp was reclaimed and an area of about 3000 ha. was prepared for cultivation. The soil of the dried swamp is a low moor peat containing 30-80% organic matter and a C/N ratio of 10-15. As a result of the draining a steady surface elevation loss of 8-10 cm/year was observed (Shoham, Levin 1968, Levin, Shoham 1983). The subsidence of the peat soil, similar to other areas in the world (Weir 1950, Stepheus 1955), was due to the consolidation and shrinkage of the low-density organic soil, to rapid oxidation of the exposed peat-soil and to wind and water erosion processes. As a result of the oxidation and aerobic micro-organism activity, extensive nitrification of the organic matter occurred, due to the low C/N ratio of the organic matter, causing an accumulation of nitrates in the upper part of the soil profile. These nitrates, leached during the rainy season to the drainage ditches and eventually to the Kinneret Lake, endanger the quality of the Kinneret water and may cause the eutrophication of the lake, which is Israel's most important drinking and irrigation water reservoir. In order to investigate the possibility of reducing the rate of decomposition of the organic matter of the peat-soil and thus minimize the nitrate formation, perennial forage crops, such as alfalfa and Rhodes grass, were planted. The results of the investigations (Levin & Leshem, 1978; Levanon, et al 1982) showed a considerable reduction of nitrate formation and accumulation under the perennial crops. In addition, the covering of the area by the perennial plants all the year round prevented soil erosion and pointed to a reduction in the subsidence rate (Levin, Shoham 1983). In the following research report, the perennial forage crops Guinea grass (Panicum Maximum) and Penisetum (cross of Pensilaria x Elephant Grass), known for their high yields and their high nutritive value as animal feed were investigated (Minson 1971).

Table 1. Guinea grass Varieties Yields, in Ton/Hectare

Cutting Date	Total	5.9.81	27.7.81	14.6.81	10.5.81	12.3.81
Days from Former Cut		40	43	35	59	
			Tift 25			
Fresh Weight	202.8	29.72	57.70	43.80	38.92	32.75
Dry Weight	44.0	6.82	13.55	7.34	7.76	8.48
Digestible Dry Weight	28.2	4.51	8.35	4.96	5.52	4.95
Total Protein %		15.7	11.6	14.7	14.8	12.25
			Gatton			
Fresh Weight	191	27.50	46.20	47.95	41.50	28.12
Dry Weight	42.4	6.32	12.22	7.80	8.89	7.22
Digestible Dry Weight	26.8	4.03	7.47	5.25	5.97	4.95
Total Protein %		12.8	12.1	14.7	13.0	13.1
			Tricheogloma			
Fresh Weight	152	34.81	43.34	30.27	39.36	19.50
Dry Weight	35.1	5.01	12.62	4.84	7.95	4.68
Digestible Dry Weight	22.0	3.29	7.81	3.22	5.36	2.36
Total Protein %		16.75	11.1	14.9	15.5	10.5

The effect of these crops as well as alfalfa, Rhodes grass and cotton on nitrogen transformations in peat-soil were recorded. The purpose of this research is twofold: a) to determine the forage production capacity, on peat-soil in the Hula Valley; b) to determine the effect of forage crops on the decomposition process of the peat and the accumulation of ammonium and nitrate ions in the soil.

Materials and Methods

In 1980 we planted forage crops at the experimental farm of the Upper Galilee Regional Council, on peat soil in the Hula Valley. The soil is mineral peat: 16% organic matter, pH - 7.2. The forage crops planted were: Guinea grass (Panicum maximum), in three varieties: 1) Tricheogloma (the variety commonly grown in Israel); 2) Gatton, from Australia; 3) Tift 25 from the experimental station near Tifton, Georgia, U.S.A.; and three crosses of Penisetum, (a cross hybrid of Pensilaria and Elephant grass), numbered 1, 2 and 3, which came from the experimental station near Tifton, Georgia, U.S.A. Yields of alfalfa, Rhodes grass, on commercial scale plots were also recorded.

Analysis of plant material: The cutting was done with a crusher harvester. The yield of a 4 x 3 metre plot in each replication was collected and weighed separately after each cutting. We took samples from each plot to determine the percentage of dry matter. Dehydrated samples were sent to be tested for digestibility in an artificial rumen and for protein content. (The tests were carried out in the laboratories of the forage crops department at the Volcani Institute, Bet Dagan). There were five cuttings of Guinea grass.

Table 2. Yields of Three Penisetum Crosses (1981), in Tons/Hectare

Digestible Dry Weight	Dry Weight	Fresh Weight	Cross
	Cut 20/8/81		
5.11	7.50	82.50	Cross 1
3.91	5.50	47.37	Cross 2
7.84	11.75	96.25	Cross 3
5.62	8.25	75.37	Average
	Cut 11/10/81		
5.68	8.57	69.56	Cross 1
6.70	9.93	93.09	Cross 2
6.25	9.39	91.87	Cross 3
6.21	9.30	84.84	Average
	Total		
10.79	16.07	152.06	Cross 1
10.67	15.43	140.46	Cross 2
14.09	21.14	188.12	Cross 3
11.83	17.55	160.21	Average

Table 3. Effect of Peat-Soil Cropping on Fungal Population

Crop	Propagules 10^3 g^{-1}
Alfalfa	Location 1. - 3^a
Alfalfa	Location 2. - 2^a
Alfalfa	Location 3. - 5^a
Cotton	- 22^b
Guinea grass	- 24^b
Rhodes grass	- 28^b
Wheat	- 27^b
Fallow peat-soil	- 25^b

Numbers followed by the same letter do not differ significantly at P=0.05.

High canopy was left in October for the winter, so as to prevent frost damage. The Penisetum was cut twice during this season.

Sampling of the soil was done once a month in the following locations:
1) on plots with no vegetation, irrigated the same as the herbage crops;
2) on a plot with no vegetation and no irrigation;
3) on plots of herbage crops;
4) on a plot of cotton,

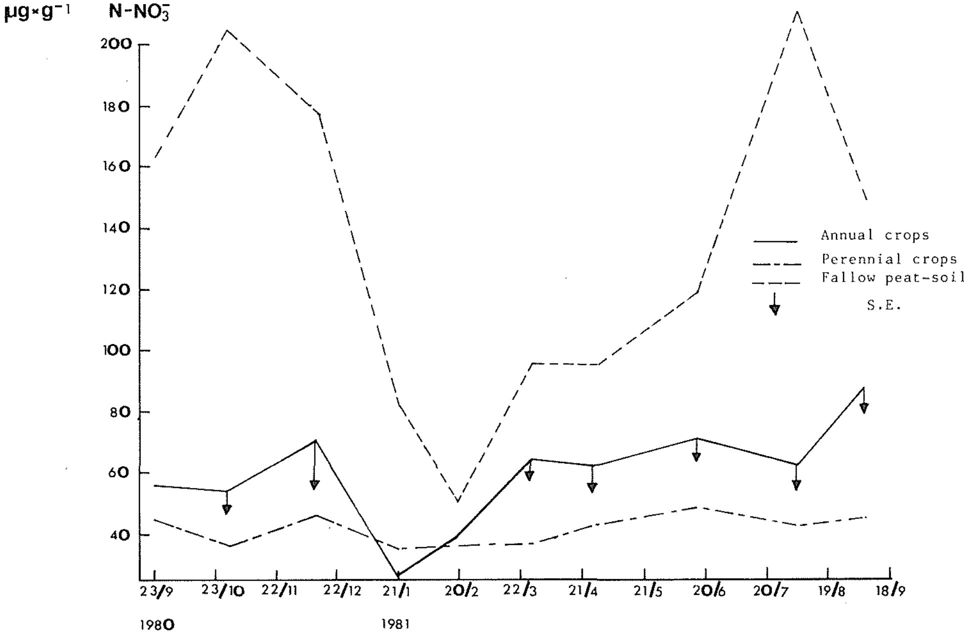

Fig 1. Nitrate concentration in peat soil at a depth of 0-30 cm

At each place samples were taken from six borings going down to 120 cm., at six different levels: 0-15 cm., 15-30 cm., 30-45 cm., 60-90 cm., and 90-120 cm. The samples were tested for N-NH$_4^+$ and N-NO$_3^-$ content in the soil. Soil microflora was counted in each location in the 0-30 cm level.

Results

Forage yields: Guinea grass: The tricheogloma variety was first to blossom, a week before the other varieties. The yields of the separate cuttings, the percentages of dry matter and the respective nutritive values are shown in Table 1.

Penisetum: The Penisetum was planted only in May, 1981. Consequently there were only two cuttings in the summer of that year. The time for cutting was determined by the height of the crop (which reached 1.5 - 2 metres). There were no blossoming yields and digestibility in the artificial rumen is shown in Table 2.

Alfalfa and to a lesser extent Rhodes grass are grown on a commercial scale in the Hula Valley. Their yields are 25-30 ton/ha. and 30-35 ton/ha. respectively. There were no differences in bacteria and actinomycetes population under the perennial crops and cotton. (10^8 and 10^6 per gramme peat-soil respectively) a significant reduction of soil's fungal population were recorded under alfalfa (table 3).

Soil Analysis: The results of the analysis for concentration of nitrate (N-NO$_3^-$) in the soil are shown in Figure 1 (0-30 cm. layer), Figure 2 (30-60 cm. layer) and Figure 3 (60-120 cm. layers).

The concentrations of ammonium ions in the soil were similar under all treatments (60-90 ppm of NH$_4^+$ -N). Likewise, there was no change in the ammonium level in the soil throughout the year.

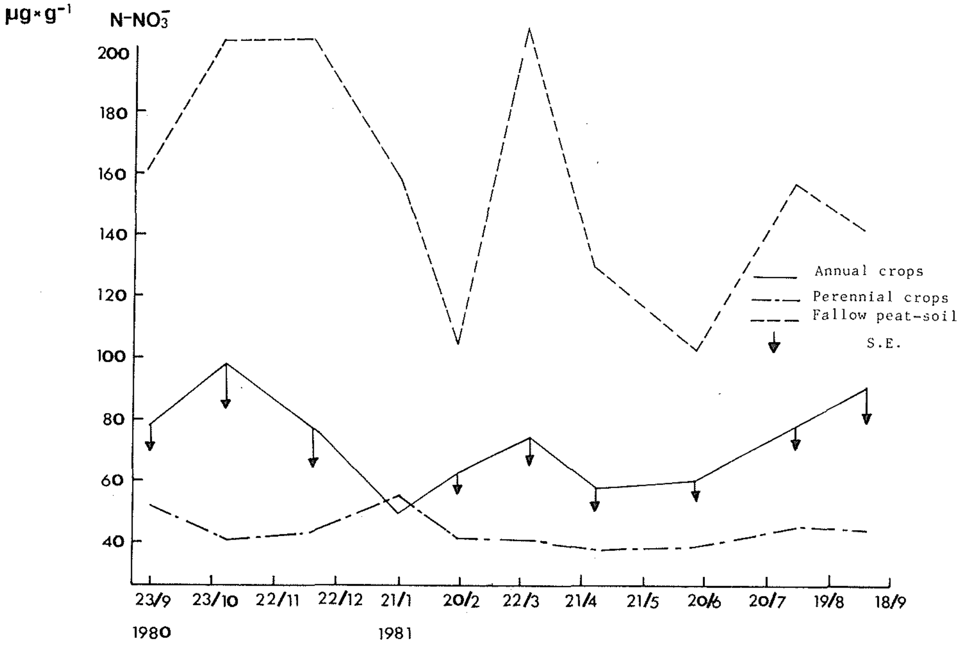

Fig2. Nitrate concentration in peat soil at a depth of 30-60 cm

Discussion

Guinea grass (Table 1): This experiment shows that by means of this crop it is possible to break through the barrier of 40 tons of dry matter per hecta a year. The two new varieties, particularly the Tift 25, surpass the common variety (tricheogloma) in yields by about 20-25%. Their digestibility is als higher by about 1-2%.

We have planned a number of plots of Guinea grass on a commercial scale in problematic areas of the Hula peat land. On these plots we shall test the yields under commercial conditions.

Penisetum (Table 2): Even though the yield was incomplete, one can disce a difference between the three crosses (Cross No. 3 has a higher yield). In view of the fact that the penisetum was cut before the blossoms appeared, its digestibility is higher than that of the Guinea grass varieties. The high yields and nutritive values of the newly introduced forage crops (especially Guinea grass) together with a growing demand for alfalfa and high quality hay is making our goal of finding economical competetive perennial crops for the Hula Valley to be achieved.

Level of NO_3^- - N in the Soil under Forage Crops and under Cotton: The organic soil in the Hula undergoes oxidation under the conditions of temperature, aeration and moisture which prevail in the field. Since the C/N ration in the organic matter of peat-soil is 1:10 (Raveh and Avnimelech 1973) and the required ratio for the activation of aerobic micro-organisms is 1:25 (Levanon 1982), a surplus of nitrogen accumulates and appears in NH_4^+ -N form, and in the course of time it turns into nitrates due to nitrification.

The nitrates are affected by the flow of rain and irrigation waters down-

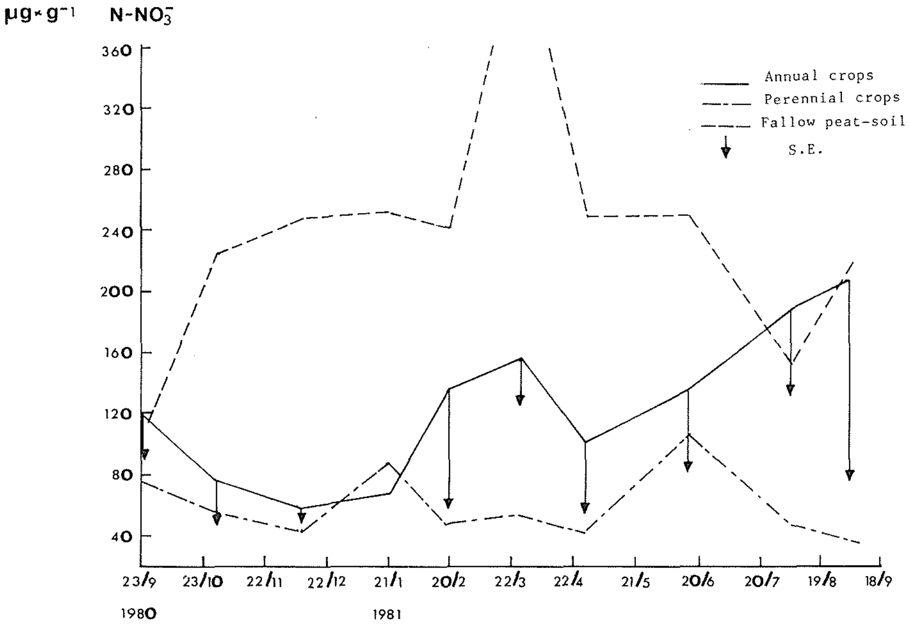

Fig 3. Nitrate concentration in peat soil at a depth of 60-120 cm

wards and the evaporation of ground waters upwards. When the soil is not covered with vegetation, concentrations of nitrates develop in the upper layer (the most aerated one), at a depth of 0-30 cm. (Fig 1). This development occurs from the end of February (when the heavy rains come to an end) until the end of the summer, when nitrate levels begin to drop due to washing by the winter rains. The picture is not so clear-cut at the 30-60 cm. level. At this level there are nitrates that were washed down from the upper layer at the beginning of the winter, as well as those that came from the rise of the level of the ground waters and those created in this layer itself (Fig 2).

From the point of view of the variations in the concentration of nitrates, the deep level (60-120 cm.) is of special interest (Fig 3). The amount of nitrates created in this level itself is minimal, because of lack of air. The changes are apparently the result of the movement of water, which bring nitrates from the upper levels due to washing caused by the winter rains. This causes a rise in nitrate concentration in the winter, whereas in the summer months there is a decline in nitrate concentration due to drainage or denitrification.

Generally one may note that a high level of nitrate concentration is preserved throughout the year under conditions of no cultivation and no irrigation during the summer months. The average concentration for the entire depth (0-120 cm.) varies between 160 and 200 ppm, namely 1200-1400 kg. of NO_3^--N per hectare.

In the plots under forage crops the nitrate concentration is very low as compared to that in an uncultivated area or under cotton. This holds true for all depths (Figs, 1-3). The low concentrations are almost constant during the entire year. This shows that here there is no gradual exploitation of the existing reservoir of nitrates, but there is a marked effect of reduction in the accumulation of nitrates due to the roots of the crops. This phenomenon parallels the findings in previous studies of cultivation of other perennial forage crops on peat soil (Levin & Leshem 1978). The reduction of nitrates under forage crops in peat soil was explained by both nitrogen consumption of the crop and an inhibitory effect on the decomposition of organic matter in soils (Levanon, et al 1979; 1982). Guinea grass, which yielded 43 tons of dry matter per hectare, removed from the soil over 1000 kg/hectare of nitrogen (table 1). The $N\text{-}NH_4^+$ concentration at all levels in all the trials was almost constant throughout the year, ranging from 60 to 90 ppm. Similar results were obtained in previous studies (Levanon, et al 1982 and Raveh, 1973)

There seems to be a dynamic equilibrium between the creation of nitrates and the creation of ammonium, whereby any addition of ammonium (aerobic decomposition of the organic matter) brings about the additional creation of nitrates, and in this manner the ammonium level in the soil is almost constant. Microflora studies of peat-soils under the tested crops showed a significant decrease in fungal population under alfalfa. These results confirmed further investigations that showed the fungistatic activity, of alfalfa root saponins on soil fungi. Soil fungi play a dominant role in peat decomposition under conditions of drying and wetting that prevail in irrigated fields (Levanon, et al 1979, 1982). Therefore from the point of view of peat-soil preservation growing alfalfa is preferable to the other perennial herbage crops, because of the decrease in biodegradation caused by soil fungi.

In conclusion we may note the following: 1) The accumulation of nitrates in soil under perennial crops were reduced drastically as compared to uncultivated soil and were significantly lower than under cotton.
2) Growing these crops and obtaining high yields of forage is possible without adding any fertilizer nitrogen.
3) High yields of good quality forage crops (43 tons of dry matter/hectare/ year) gives the farmers of the Hula Valley the possibility of growing a crop with both economic and ecologic advantages.
4) The cultivation of perennial herbage crops, especially alfalfa shows a possible way of preserving and minimizing a subsidence of a problematic, decomposing organic soil.

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ORIGIN AND TREATMENT OF HYDROCOMPACTION IN THE SAN JOAQUIN VALLEY, CA, USA

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Abstract

Hydrocompaction is a property of some dry sediments which causes them to spontaneously slump, crack and collapse after wetting. In California's San Joaquin Valley, hydrocompaction with vertical displacements up to 5 m and cracks up to 2 m wide has spotty occurrences in Pleistocene mudflow deposits. After completion of the process, the originally hazardous deposits become stable. The probable origin of hydrocompaction in California and in some loess deposits is Pleistocene, periglacial, sublimation of near surface wet deposits, expanded by previous freezing.

Hydrocompaction in California affected two reaches of the San Luis Canal and alinements of several associated pipelines. In order to prevent postconstruction damage, 32.5 km of canal alinement and 90 km of selected pipeline alinements were flooded prior to construction, at a cost of some 8 million dollars. No failures have occurred in prewetted areas. Additional wetting of some untreated road crossings and pipeline laterals is under consideration, due to numerous failures in nonprewetted areas.

Introduction

The paper briefly summarizes data on distribution and treatment of hydrocompaction in a major irrigation project in the semiarid, west-central portion of the San Joaquin Valley, California, USA (Fig. 1). The project, known as the San Luis Unit of the Central Valley Project, consists of several dams, canals, underground distribution pipelines, and associated structures (Anonymous, 1981). Bureau studies of hydrocompaction included delineation of areas potentially susceptible to hydrocompaction, estimates of ultimate amounts of hydrocompaction, selection of methods to control the destructive effects of hydrocompaction, preconstruction treatment of selected alinements and evaluation of the efficiency of this treatment. The text is based mostly on data collected during two decades of preconstruction, construction and postconstruction studies by the Federal Bureau of Reclamation. The ideas expressed in this paper are, however, those of the author and may not represent the official view of the Bureau.

Hydrocompaction - General Data

Hydrocompaction occurs in some dry, unconsolidated, porous semiarid, and arid deposits when they lose their dry strength after wetting (sometimes rapidly), and develop spontaneous settling, slumpage, and cracking. The process can be one of the most rapid and highly destructive forms of subsidence. Typical surface expressions of active hydrocompaction are soil cracks and settling, development of "sinkholes" similar to karst sinks, undulatory fields, piping, and structural damage to houses, industrial and agricultural structures, railroads, highways, and bridges. Particularly severe damage occurs to dams, canals, ditches and wells (Fig. 2). The process is frequently related to human activity, such as irrigation, construction of canals, urbanization, disposal of industrial wastewater, etc.

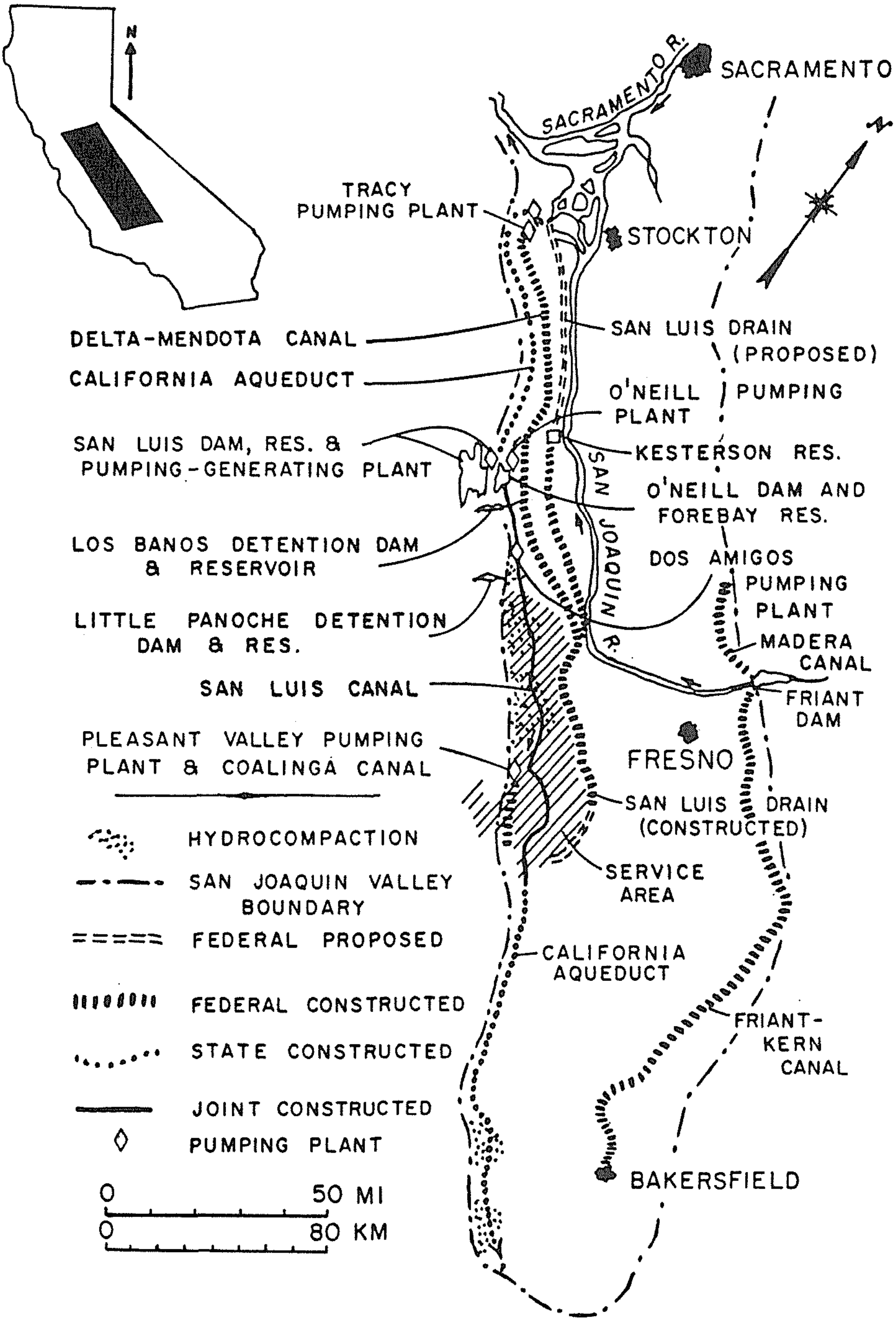

Fig. 1. San Luis Unit and other major Federal and State conveyance systems in the San Joaqin Valley, California.

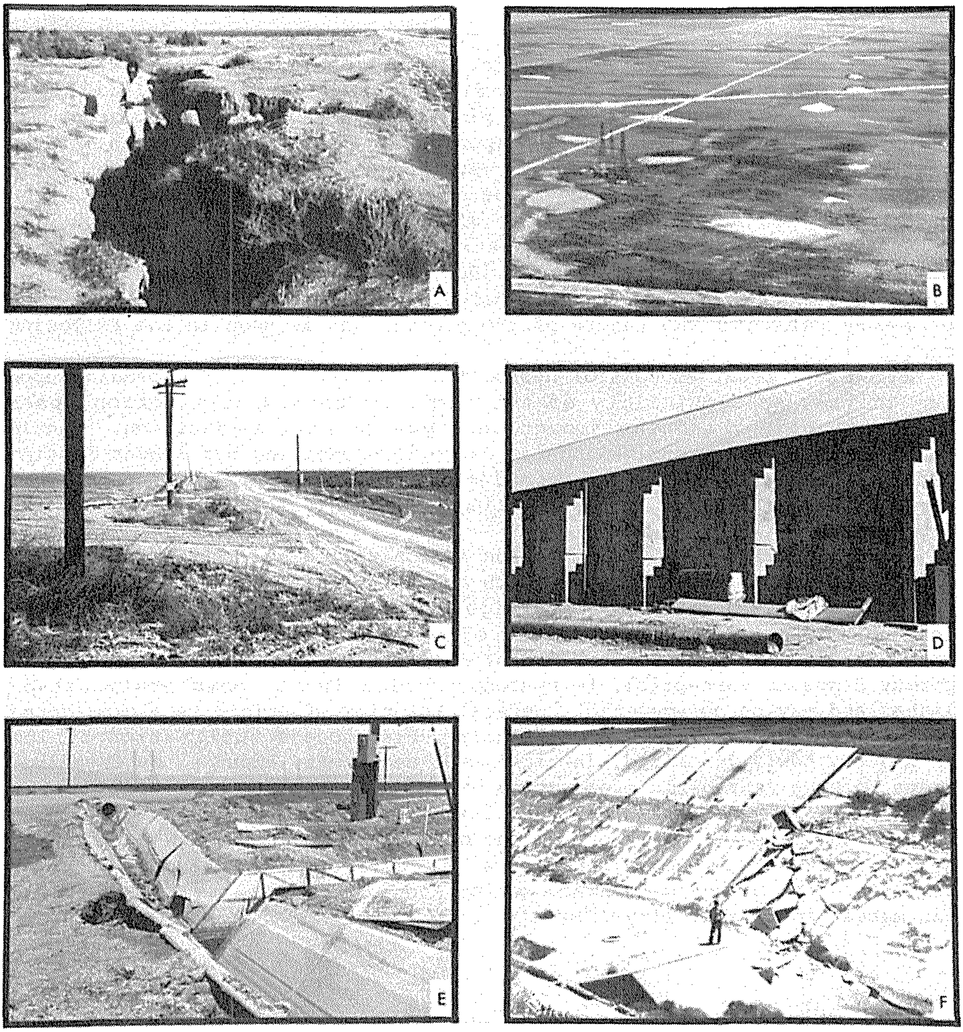

Fig. 2. Typical damage due to hydrocompaction in the San Joaquin Valley: (A) Hydrocompaction cracks. (B) 3-4 m deep "sink holes", partially flooded by irrigation runoff in an originally flat leveled field. Note high-power line for scale. (C) An originally flat field-road became undulatory by hydrocompaction due to irrigation. (D) Cracking of a farm workshop due to irrigation of surrounding fields. (E) Cracking and separation of concrete lining of a small irrigation ditch. (F) Damages to the 10-cm-thick concrete lining in a test canal section (State of California, "Mendota Test Site").

Hydrocompaction is known to exist in several areas of the United States and abroad (Anonymous, 1959, 1963; Bull, 1961, 1964; Drashevska, 1962; Dudley, 1970; Lin and Liang, 1980; Lofgren,1969). The phenomenon is described under various terms such as "shallow subsidence", "near-surface subsidence", "hydroconsolidation", "soil settling", "soil settling by wetting", "collapsing soils", etc. Some of these terms are misleading. For example, the term "shallow subsidence", previously used in the San Joaquin Valley (Poland, 1958), is misleading because (1) the process may not be restricted to near-surface deposits and can occur at depths of over 30-40 m and (2) the uppermost near-surface deposits may be stable, while deposits occurring at depth can be susceptible to hydrocompaction. The term, "hydroconsolidation", can be associated with some form of consolidation by cementation, etc. The term "hydrocompaction", used by the Bureau of Reclamation (Anonymous, 1963; Hall and Carlson, 1965; Prokopovich, 1963) and by the U.S. Geological Survey (Lofgren, 1969), and others properly reflects the nature of the process and is used in the following text.

Rates and total amounts of hydrocompaction vary greatly depending upon the initial susceptibility of the sediment to hydrocompaction, past hydrocompaction, and the amount and type of water application. Gentle water application by sprinkling may cause less damaging but longer lasting hydrocompaction. After the completion of hydrocompaction, sometimes several years after the initial wetting, the originally hazardous deposits become stable.

Susceptibility to hydrocompaction is controlled by (1) the development of an excessively porous deposit with a low dry density and a low moisture content, but with a relatively high dry strength which prevents spontaneous compaction and (2) aridity of past and present climates which prevents "natural hydrocompaction" of deposits. The origin of excessively porous deposits susceptible to hydrocompaction is not completely established and may be polygenetic. Several theories of origin were considered for the San Joaquin Valley. Periglacial, Pleistocene sublimation of ice in frozen, newly deposited, initially wet sediments, expanded by freezing, seems to be a reasonable explanation of the origin of hydrocompaction in California's San Joaquin Valley and in many loess deposits in both North America and Eurasia. Such origin was duplicated in laboratory experiments by the author. In California, the "freeze-drying" was accomplished by westerly winds bringing downward moving and, therefore, relatively dry, air masses into the Valley (Fig. 3).

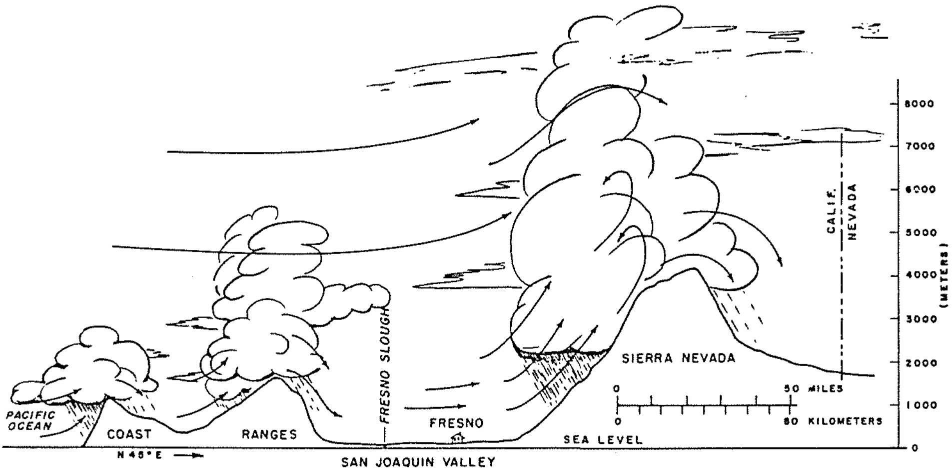

Fig. 3. Air circulation patterns in the San Joaquin Valley, California.

Hydrocompaction in California

In California, at the present time, hydrocompaction has "spotty" occurrence along the Coast Range foothills on the western margin of the southwestern portion of the arid San Joaquin Valley (Fig. 4), where it affects two major water conveyance systems--the San Luis Canal and the California Aqueduct. Typical deposits here, affected by hydrocompaction, are composed of clayey Pleistocene interfan piedmont alluvium deposited mostly as mudflows. Such interfans are composed of minor, coalescent poorly defined fans of small arroyos. These isolated patches of alluvium, susceptible to hydrocompaction, are separated by fluvial alluvium of major ephemeral Coast Range streams (Fig. 4). The maximum amount of hydrocompaction here is up to 5 m. Some hydrocompaction cracks were over 2 m wide and were probed to a depth of about 10 m.

The initial total thickness of sediments susceptible to hydrocompaction was in an order of 65 m. At the present time, much of the alluvium initially susceptible to hydrocompaction has become "stable" due to wetting caused by irrigation. Since the 1940's, with advanced farming and widespread irrigation, hydrocompaction caused extensive damages to irrigation wells, ditches, pipelines, roads, bridges, fields and buildings (Fig. 2).

Hydrocompaction in the region was probably first noted in 1915, after construction of the now abandoned Chaney Pumping Station in an undeveloped terrain (Sneddon, 1951). Hydrocompaction here was kept well under control for decades by keeping the station perimeter dry through diversion of wastewater and by restriction of irrigation. Since 1955-60, the hydrocompaction has become particularly important due to scheduled construction of the San Luis Canal and its distribution system by the Federal Bureau of Reclamation, and associated developments of the State Water Project (Anonymous, 1974, 1974A).

U.S. Bureau of Reclamation Studies of Hydrocompaction

U.S.B.R. studies of hydrocompaction, initially carried out as part of an interagency investigation of subsidence in California (Poland, 1958), were accelerated since 1961-62, as engineering-geologic preconstruction, construction and postconstruction studies of hydrocompaction. These studies included (1) evaluation of numerous proposed methods of detection of sediments susceptible to hydrocompaction (Prokopovich, 1963, 1984), (2) estimates of ultimate amounts of future hydrocompaction, and (3) selection of susceptible areas for preconstruction treatment and monitoring of its effectiveness.

Field exploration included periodic mapping of hydrocompaction features such as cracks, sinks, damaged structures, and road conditions; collecting data on past irrigation; drilling of over 600 test holes, collecting of over 7,400 "undisturbed" core samples for laboratory studies; and operation of several artificially control-flooded, closely monitored ponds.

Probably the best method for detection of sediments susceptible to hydrocompaction is the evaluation of laboratory consolidometer tests, conducted on naturally moist and water-saturated core samples (Lin and Liang, 1980). This approach, however, was not used initially due to both the lack of consolidometers in field laboratories, and the necessity of conducting many time-consuming tests. Because of widespread irrigation at the time of canal construction, a direct evaluation of field observations indicating the presence or absence of hydrocompaction in the area was more feasible (Fig. 5). Numerous X-ray diffraction and differential thermal analyses, and chemical studies of soluble salts, were also included in the studies.

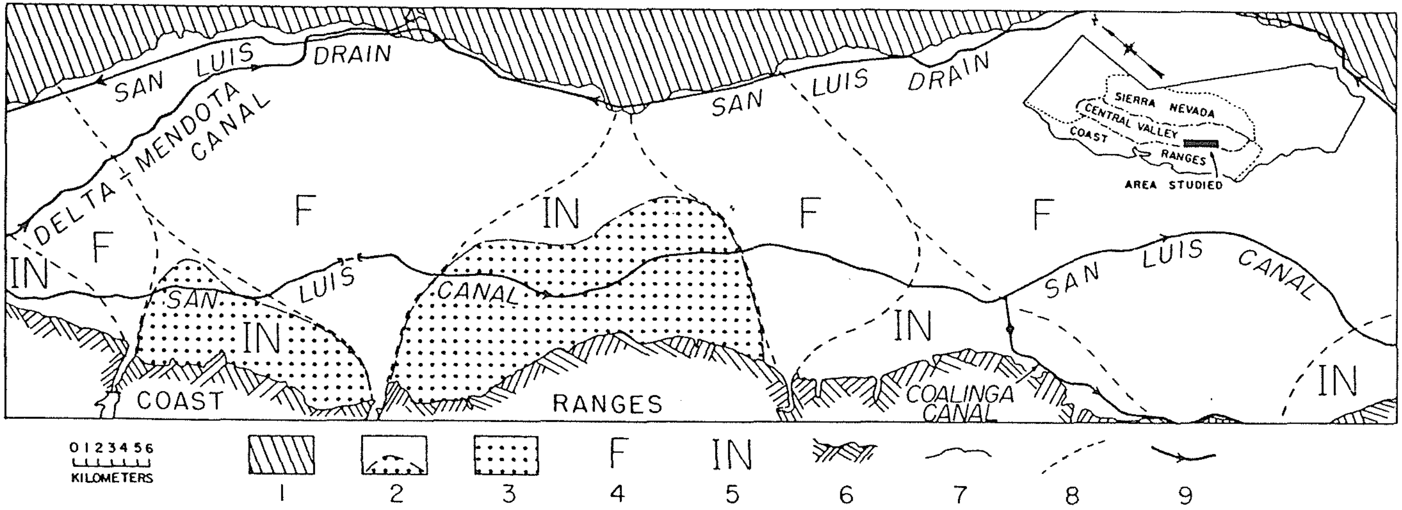

Fig. 4. Generalized geologic map of a portion of San Luis service area and vicinity, showing: (1) Basin alluvium; (2) Westside or Coast Range Piedmont alluvium; (3) Area affected by hydrocompaction; (4) Major alluvial fan; (5) Interfan; (6) Coast Range foothills; (7) Geologic contact; (8) Boundary of fans and interfans; and (9) U.S. Bureau of Reclamation canals.

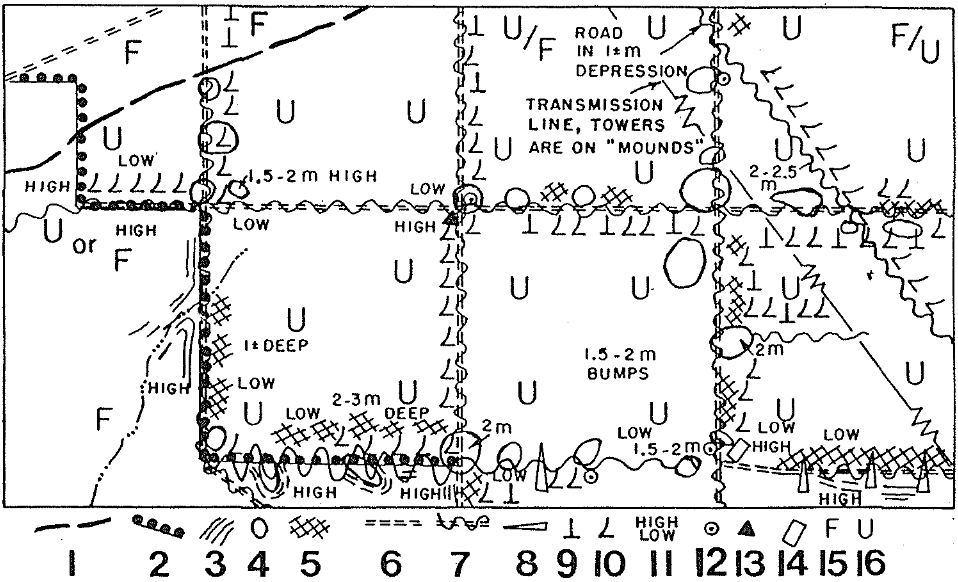

Fig. 5. An example of hydrocompaction mapping. 1. Boundary of an area affected by hydrocompaction. 2. Boundary between irrigated and "virgin" land. 3. Cracks. 4. Sinks. 5. Troughs. 6. Flat and 7. bumpy roads. 8. Tilted road. 9. Straight and 10. tilted powerline poles. 11. Difference in field level. 12. Irrigation well. 13. Tailwater pond. 14. Tilted building. 15. Flat field and 16. undulatory field.

U.S.B.R. Treatment of Hydrocompaction

Severe damage by hydrocompaction can be averted by (1) keeping the hazardous terrain dry or 2) by consolidation of this terrain by preconstruction wetting. The first approach was successfully used for decades at Chaney Pumping Station. For obvious reasons, such an approach is not applicable to a large water conveyance system such as the San Luis Canal and its distribution system, which are aimed at irrigating the surrounding land. Preconstruction flooding was adopted, therefore, as the main method of averting hydrocompaction along the San Luis Canal (Anonymous, 1974) and its distribution system. Similar flooding at a cost of some $8.5 million was also adopted by the State of California in the southern part of San Joaquin Valley during construction of the portion of the California Aqueduct south of the San Luis Canal (Anonymous 1974A).

Bureau's prewetting, using a system of shallow, rectangular ponds, separated by low earth dikes (Fig. 6A,B,C,D) was conducted in two reaches of the San Luis Canal, totalling some 32.5 km. A total of 20 ponds were constructed in the 8.5-km-long northern reach, and another 108 ponds were constructed further downstream, in the 24-km-long southern reach. As no canal water was available for ponding during prewetting, pump lifted water had to be delivered via specially constructed feeder ditches and pipelines. To speed water penetration into soil, gypsum was added in some test ponds. Gravel-packed infiltration wells; 23, 30 and 38 m deep; were constructed in subreaches with particularly dry soil in order to speed water infiltration. Plastic covers were installed in places to prevent wind-wave erosion of the

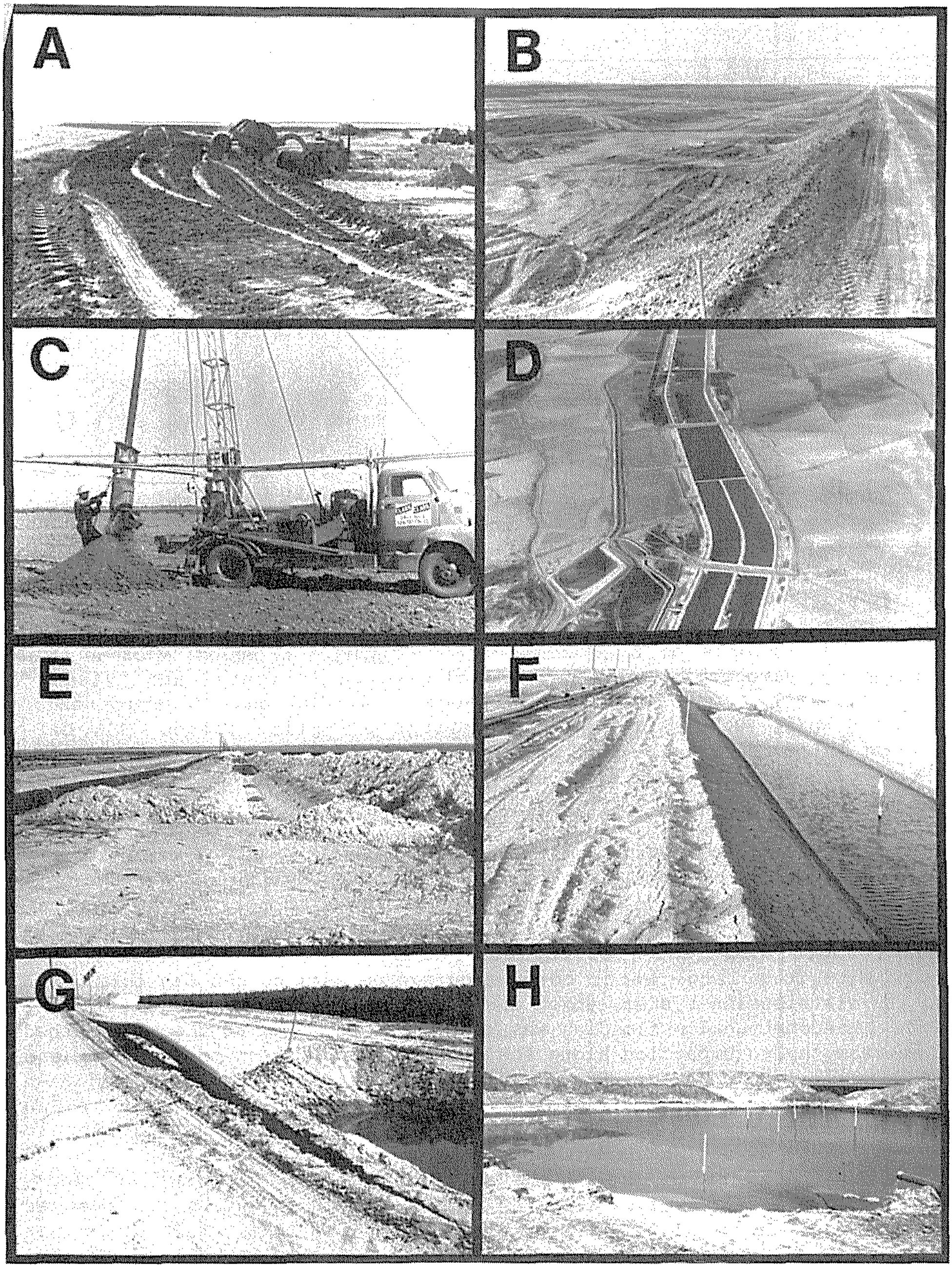

Fig. 6. Treatment of hydrocompacton along San Luis Canal (A, B, C, D), and pipeline alinements (E, F, G, H). (A) Excavation of preconsolidation ponds. (B) Ponds prior to flooding. (C) Drilling of infiltration wells (D) Flooded preconsolidation ponds. (E) Trench prior to flooding. (F) Flooded trench. (G) Cracks due to hydrocompaction. (H) Preconsolidation pond at pumping plant site.

dikes. Numerous surface and subsurface bench marks were installed in ponds and periodically releveled in order to monitor the progress of hydrocompaction. Several test holes, up to 60 m deep, were drilled in ponded areas during and after wetting in order to monitor moisture-density changes of soils. After 12 to 18 months of flooding, the ponds were allowed to dry for 5 to 6 months. The amount of hydrocompaction achieved in individual ponds was erratic and ranged from traces to 2.4 m. The treatment was highly effective and no significant canal lining failures caused by hydrocompaction have been observed in the treated areas, i.e., 15 years after construction. Some minor, local undulation and local cracking of the concrete lining by hydrocompaction was noted, however, in a few areas outside of the prewetted reaches. These minor damages did not interfere with canal operation. Therefore, the quality of preconstruction delineation of the extent of hydrocompaction along the canal was reasonably good.

Among 1550-km-long alinements of San Luis underground distribution pipelines, several alinements totaling 188 km are located in areas affected by or susceptible to hydrocompaction. Ninety km of these alinements were selected by the designers for preconstruction treatment. Most preconsolidation was accomplished by flooding. Gravel-packed infiltration wells were operated in critical subreaches and several, large, rectangular ponds were constructed at selected structure sites (Fig. 6E,F,G,H). Numerous road and pipe crossings were left untreated by designers to prevent their damage. A few reaches were preconsolidated by less expensive mechanical compaction. No pipe breaks were reported in prewetted areas, but more than 30 breaks and leaks occurred in nontreated or mechanically compacted subreaches. Future failures of several incomplete laterals in unprewetted crossings seem to be unavoidable. Additional prewetting of these crossings and other nonprewetted subreaches is now under consideration. Also suggested is gradual postconstruction wetting, from the ground water table upward of some originally not prewetted structure sites.

The contract costs of treatment of hydrocompaction in the area (Prokopovich and Marriott, 1983) are as follows:

San Luis Canal - preconsolidation.	$3,845,000
Laterals - preconsolidation.	2,629,000
Lateral repairs.	107,000
Total contract cost	6,581,000

Actual cost, including geological exploration, laboratory testing, survey control, etc. is approximately 20 percent higher, i.e., is about $8 million. The above sums, expressed in 1983 dollars, yield the total cost of hydrocompaction treatment in an order of $15 million.

Conclusions

Experience obtained during construction of the San Luis Unit indicates that: (1) postconstruction hydrocompaction can be successfully eliminated by proper preconstruction flooding; (2) consolidation by mechanical compaction is not reliable; (3) flooding is an expensive treatment, and careful economic evaluation of its cost versus cost of postconstruction repairs and other losses should be made for each alinement.

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NATURAL AND INDUCED SINKHOLE DEVELOPMENT IN THE EASTERN UNITED STATES

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Abstract

Detailed investigations of sinkhole occurrence have been previously limited to Alabama and Missouri. A reconnaissance-type investigation of this occurrence in the eastern United States was made in 1981 to regionalize previous findings. About 850 sites at which an estimated 6,000 sinkholes have occurred were identified in 19 States. States most impacted by sinkholes were Alabama, Florida, Georgia, and Tennessee in the southeast, Pennsylvania in the northeast, and Missouri in the midwest. Costs of damage and protective measures to minimize severity of potential additional sinkhole occurrence reported for limited sites was about 170 million dollars.

Sinkholes are separated into two categories, induced (accelerated or caused by man) and natural. Most induced sinkholes result from water-level declines due to ground-water withdrawals or diversions and impoundment of surface drainage. Natural sinkholes result from progressive solution of bedrock and from natural water-level declines that trigger the same mechanisms causing induced sinkholes.

INTRODUCTION

Sudden large collapses in the land surface forming new sinkholes (fig. 1) in recent years have focused attention on a little understood geologic hazard. Few realize that thousands of similar but smaller sinkholes have formed in the United States since 1950. Costly damage has resulted from their sudden collapse beneath highways, railroads, buildings, dams, reservoirs, pipelines, vehicles, and people. They have also resulted in or are potential sources of pollution of water supplies. Concern has increased with the growing awareness that many, if not most, sinkholes occurring in numerous areas are induced by man's activities and that most of these involve changes to the hydrologic environment.

Sinkholes can be separated into two categories, even though most factors involved in their occurrence are the same. These categories are defined as "induced" and "natural." Induced sinkholes are those caused or accelerated by man's activities whereas natural ones are not (Newton, 1976a). Recognition of induced sinkholes and limited investigations of them are confined to this century. Almost all investigations dealing with triggering mechanisms or processes have been made since 1950. Research has been limited and local in nature. Regional investigations have not been made.

The purpose of this paper is to evaluate the magnitude of active (measurable) sinkhole development in the eastern United States, insofar as possible, and to regionalize findings resulting from local investigations.

MAGNITUDE OF PROBLEM

Active sinkhole development has been identified in 19 of 31 States in the eastern United States. This identification is based on a search of the

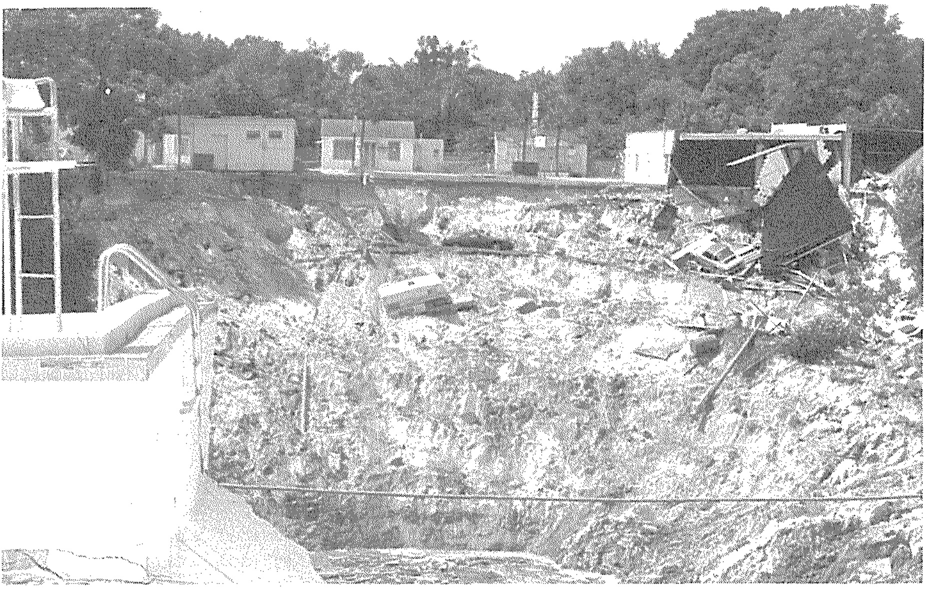


Figure 1. Collapse in Winter Park, Florida, 1981.
(Photograph by J. L. Smoot, U.S. Geological Survey)

literature and a reconnaissance-type inventory made in 1981. Significant inventories prior to 1981 were limited to those for Missouri (Aley and others, 1972; Williams and Vineyard, 1976) and Alabama (Newton, 1976a).

The total number of sites of collapses for which information is available exceeds 850 (fig. 2). Because of the density of unrelated sites in some areas, numerous locations represent groups of sites. Similarly, individual sites may represent either single or multiple related collapses. The total number of collapses occurring at all sites is estimated to exceed 6,000.

Available information shows that the occurrence and impact of sinkhole development in the eastern United States has been most significant in Alabama, Florida, Georgia, and Tennessee in the southeast, Pennsylvania in the northeast, and Missouri in the mid-west. States with the least problem are in the north in areas previously occupied or affected by ancient glaciers (fig. 2). Underground openings in many of these areas, like those in similar areas in Canada (Davies, 1951), were probably scraped away, filled with debris, or covered with glacial deposits.

Collapses forming new sinkholes commonly pose hazards to man with the degree of hazard depending more on how rapidly they form than on their size. Few fatalities and injuries have been reported. Fatalities resulting from or associated with collapses have been reported in Florida, Missouri, and Pennsylvania. Injuries were reported in Alabama, Florida, and South Carolina.

Cost of damage and protective measures to minimize severity of potential sinkhole occurrence has been significant. How significant, however, is unknown. Costs, available only for a limited number of sites, amounted to about 170 million dollars expended almost entirely after 1970. Of this, about 130 million was expended to eliminate or minimize sinkhole related problems at five dams in Alabama, Georgia, Kentucky, and North Carolina, and about 11 million to repair or protect highways in Alabama and Tennessee.

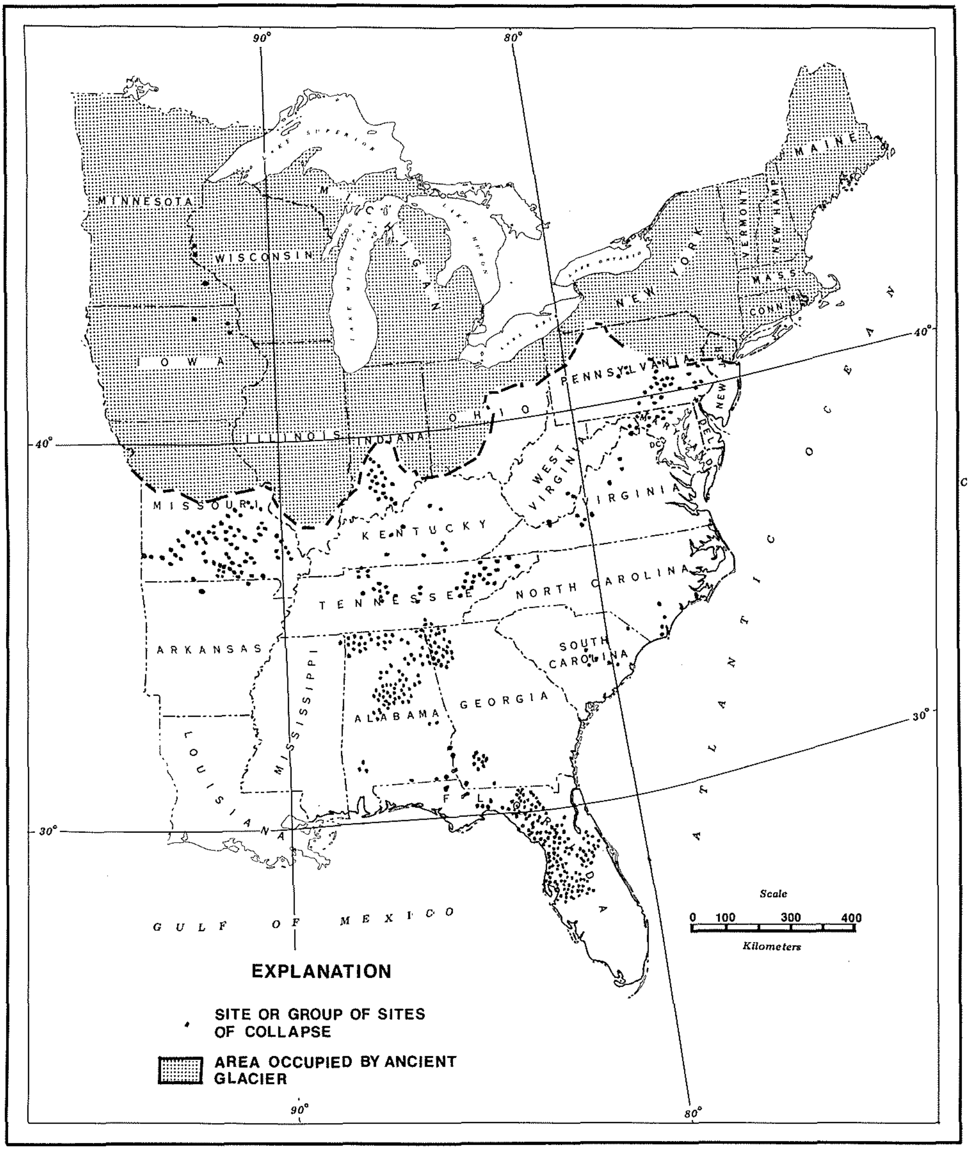
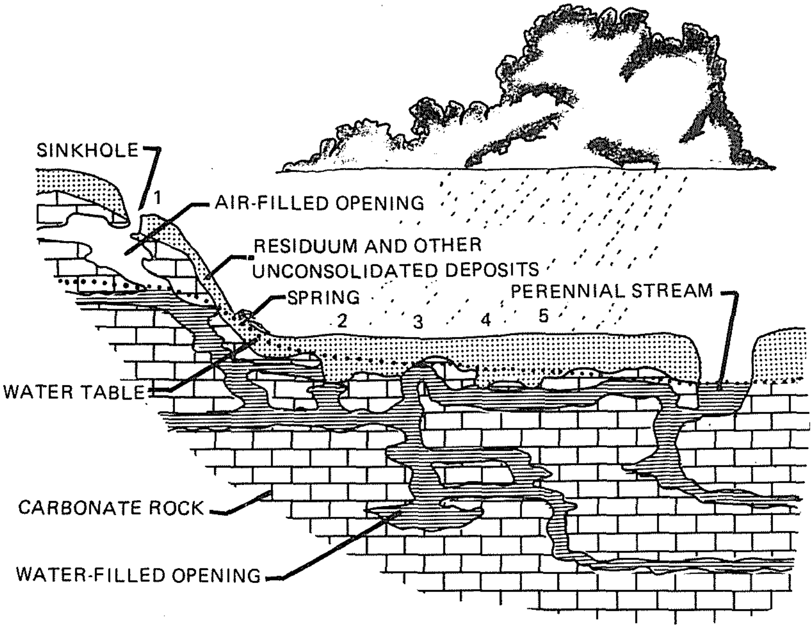


Figure 2. Sites of active sinkhole development in the eastern United States.

All new sinkholes are potential sources or avenues of pollution to underlying aquifers and nearby streams. Because of their relation to man's activities and water, collapses beneath sewers and storm drains and beneath sewage and industrial process impoundments are one of the more common occurrences. Collapses and draining of sewage lagoons into aquifers have been reported in Alabama (Warren, 1974) and in Missouri where effluent reappeared in springs and wells located as much as 2.4 km from the source (Aley and others, 1972). Similar activity beneath sewers, sewage lagoons, and



(Numbers apply to sites described in report.)

Figure 3. Schematic cross-sectional diagram of basin showing geologic and hydrologic conditions.

industrial process impoundments has occurred in Alabama, Florida, Georgia, Minnesota, Missouri, North Carolina, Pennsylvania, and Tennessee. Unfortunately, studies relating to this impact are not readily available in the literature.

GEOLOGIC AND HYDROLOGIC SETTING

The terrane used to illustrate sinkhole development, hereafter referred to as the "selected terrane", is a youthful basin underlain by carbonate rocks such as limestone, dolomite, and marble (fig. 3). It contains a perennial or near-perennial stream. Water is stored in underlying rocks and moves through interconnected openings along bedding planes, joints, fractures, and faults that often are enlarged by solutioning. It moves in response to gravity, generally toward the stream channel where it discharges and becomes streamflow.

Water in carbonate rocks occurs under water-table and artesian conditions; however, this study is concerned primarily with water-table conditions. The configuration of the water table conforms to that of the topography but is influenced by geologic structure, water withdrawal, and precipitation. The lowest water level occurs where the water table intersects the stream channel (fig. 3). Bedrock openings underlying lower parts of the basin are water filled and those underlying highland areas are air filled.

A mantle of unconsolidated deposits resulting from the solution of the underlying rocks, consists chiefly of residual clay (residuum). This clay, commonly containing chert debris, covers most bedrock. Alluvial or other unconsolidated deposits often overlie the clay. Other unconsolidated deposits commonly fill openings in bedrock. The contact between residuum and underlying bedrock, because of differential solution, can be highly irregular (fig. 3).

SINKHOLE DEVELOPMENT

Sudden occurrence of natural and induced sinkholes results from (1) collapse of the roof of a cavity or cavern in rock due to its progressive enlargement by solution or (2) the downward migration of soil and other unconsolidated deposits into openings in the top of bedrock (Newton, 1976a). Collapse of bedrock roofs, in comparison to the migration of unconsolidated deposits into underlying openings, is rare. According to Williams and Vineyard (1976) in a study involving 97 collapses in Missouri, "Although cavern roof collapse in bedrock has been cited as a cause of catastrophic sinkhole formation, no contemporary event that could be attributed directly to this cause has been observed." Similar observations have been made in Tennessee (Moore, 1980), Alabama (Newton and Hyde, 1971) and elsewhere. Of hundreds of collapses observed by the author, only one could be classified as a possible collapse of the roof of an opening in bedrock (Newton, 1976a).

Most collapses forming sinkholes result from roof failures of cavities in unconsolidated deposits overlying carbonate rocks. These cavities are created when the unconsolidated deposits migrate or are eroded downward into openings in the top of bedrock. When this occurs, a void is created. Their occurrence, growth, and collapse has been described by Donaldson (1963), Jennings and others (1965), and many others.

The typical cavity in unconsolidated deposits is circular with the configuration of the top resembling a dome or arch. The sides at the bottom generally coincide with pinnacles or irregularities in the top of bedrock and the walls are usually vertical as the opening grows toward the land surface (fig. 4A). This configuration, however, can be modified by the shape of the underlying opening in bedrock and by variations in the cohesion or competence of overlying beds. The configuration of the typical cavity can change when its upward growth reaches a more competent bed. The roof flattens, the growth continues laterally, and the walls taper toward the opening in bedrock (fig. 4B). Other cavity configurations such as horizontal tunneling along the top of bedrock has also been observed (Newton, 1976b).

A major difference in natural and induced sinkholes is the time required for their development. Some induced sinkholes develop within hours after the effects of man's activities are exerted on existing geologic and hydrologic conditions. In contrast, the development of a natural sinkhole may require tens, hundreds, or even thousands of years. Development of all sinkholes, regardless of their category, is dependent on the solution of bedrock.

The classification of sinkholes has one major advantage. The number of induced collapses in numerous areas allows the investigator to observe and record factors or mechanisms involved in initial stages of development. Comparable information, because of their rare occurrence in many terranes, is not available to evaluate initial stages of development of natural sinkholes.

INDUCED SINKHOLES

Induced sinkholes were first classified by separating those caused by lowering the water level from those caused by raising the water level (Aley and others, 1972). This classification was modified slightly by Newton (1976a) by separating those caused by declines in water levels due to groundwater withdrawals from those caused by construction. Collapses resulting

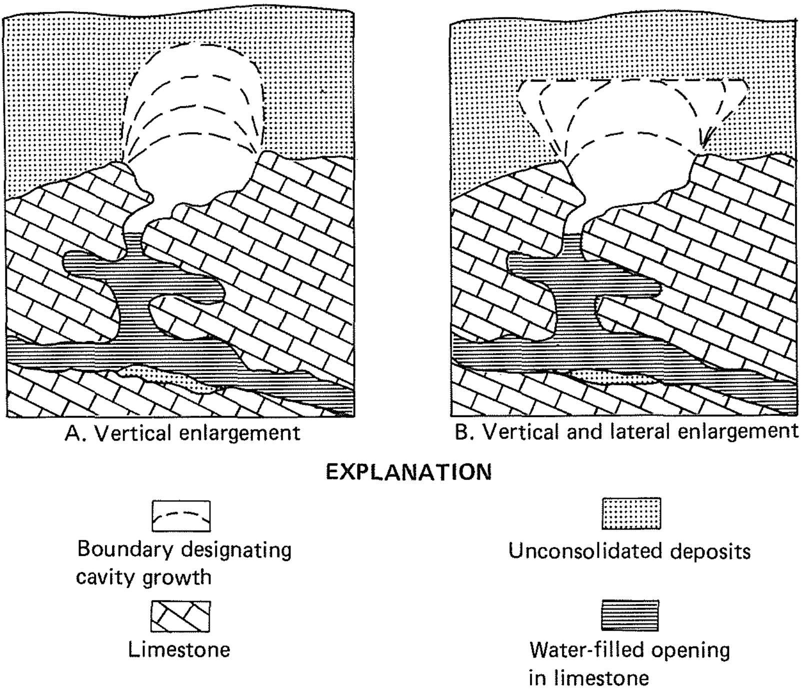


Figure 4. Development of cavities in unconsolidated deposits. (Modified from Newton, 1976a)

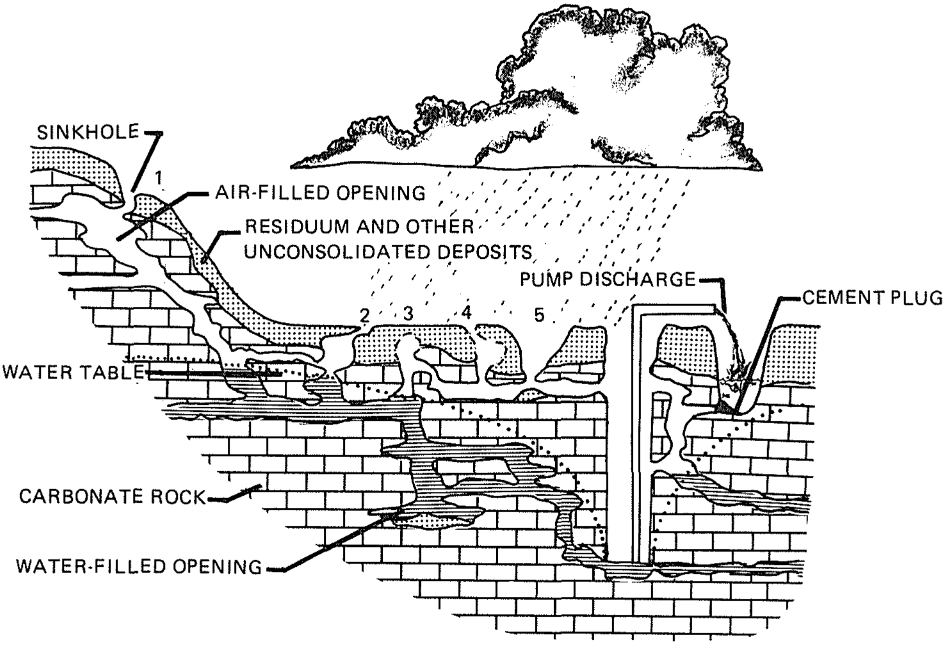
from construction, as used here, include those caused by the erection of a structure, the impounding or diverting of surface water, and any modification of the land surface.

Decline of Water Level

Foose (1953), in the first investigation of this type sinkhole activity, associated the occurrence of sinkholes with pumping and a subsequent decline in the water table. He determined that their formation was confined to areas where a drastic lowering of the water table had occurred, that their occurrence ceased when the water table recovered, and that the shape of collapses indicated a lowering of the water table and withdrawal of support. Robinson and others (1953) attributed sinkhole occurrence in a cone of depression to the increased velocity of groundwater movement causing collapse of clay and rock filled cavities in bedrock.

Jennings and others (1965) associated development of sinkholes with pumpage and creation of cones of depression and determined that sinkhole and subsidence problems increased where the water table was lowered.

Spigner (1978) attributed intense sinkhole development near Jamestown, South Carolina to a water level decline resulting from pumpage, provided descriptions indicating loss of support and attributed some downward movement of unconsolidated deposits to piping. Sinclair (1982) attributed similar activity in Florida to loss of support and water-level fluctuations.



(Numbers apply to sites described in report.)

Figure 5. Schematic cross-sectional diagram of basin showing changes in geologic and hydrologic conditions resulting from water withdrawal.

Subsidence sometimes accompanies the formation of sinkholes due to declines in water level. In Alabama, it sometimes precedes collapse (Newton and Hyde, 1971). Movement of unconsolidated materials into bedrock where the strength of overlying material is not sufficient to maintain a cavity roof, will result in subsidence at the surface (Donaldson, 1963).

Cited reports have described only indirectly or in part the hydrologic mechanisms resulting from a decline that cause the downward migration of unconsolidated deposits. These mechanisms, based on studies in Alabama (Newton and Hyde, 1971; Newton and others, 1973; and Newton, 1976a) are (1) loss of buoyant support to roofs of cavities or caverns in bedrock previously filled with water and to residual clay or other unconsolidated deposits overlying openings in the top of bedrock, (2) increase in the velocity of movement of ground water, (3) increase in the amplitude of water-level fluctuations, and (4) movement of water from the land surface to openings in underlying bedrock where recharge had previously been largely rejected because they were water filled.

A cone of depression resulting from pumpage from a quarry is superimposed on a schematic diagram of the selected terrane to illustrate the downward migration of unconsolidated deposits, creation of cavities in them, and sinkhole development (fig. 5). A solutionally enlarged opening in the stream has been sealed, a common mine dewatering practice, to prevent flooding in the cone of depression.

The loss of buoyant support following the water-level decline can result in an immediate collapse of roofs of openings in bedrock and unconsolidated deposits or can cause a downward migration of unconsolidated deposits spanning openings in the top of bedrock. The buoyant support exerted by water on a solid and, hypothetically, unsaturated clay overlying an opening in bedrock, for instance, would be equal to about 40 percent of its weight.

A collapse triggered by a loss of buoyant support is illustrated at site 5 in figure 5. Loss of support also triggered the downward movement of residual clay and the creation of a cavity in unconsolidated deposits at site 3. Openings in the top of bedrock at both sites were overlain by unconsolidated deposits prior to the decline of the water table (fig. 3).

The creation of a cone of depression in an area of water withdrawal results in an increased hydraulic gradient toward the point of discharge (fig. 5) and a corresponding increase in the velocity of ground-water movement. Erosion caused by this movement through unobstructed openings and against joints, fractures, faults, or other openings filled with clay or other unconsolidated sediments results in the creation of cavities that enlarge and eventually collapse.

Pumpage results in water-level fluctuations greater in magnitude than those occurring under natural conditions. The repeated movement of water through openings in bedrock against overlying unconsolidated deposits causes repeated addition and subtraction of buoyant support to them and repeated saturation and drying. Both result in the downward migration of the deposits that creates or enlarges cavities in them. All collapses and cavities in unconsolidated deposits illustrated in figure 5 could have resulted from this mechanism.

A drastic decline of the water table in the lowland areas in the selected terrane (fig. 5) where all openings in the underlying bedrock were previously water filled results in induced recharge of surface water. This recharge was partly rejected prior to the decline because the openings were water filled. The inducement of recharge through openings in unconsolidated deposits interconnected with openings in bedrock results in the creation of cavities in the deposits. The material immediately overlying the bedrock openings is eroded to lower elevations. The water table, previously located above the top of bedrock (fig. 3), is no longer in a position to dissipate the mechanical energy of downward moving recharge. Repeated rains result in the progressive enlargement of this type cavity. A corresponding thinning of the cavity roof due to its enlargement toward the surface eventually results in collapse. The position of the water table below unconsolidated deposits and openings in the top of bedrock favorable to induced recharge is illustrated at sites 2 and 4 in figure 5. Cavities in unconsolidated deposits at these sites were formed primarily or in part by induced recharge. Surface openings and underlying cavities illustrated at the sites are identical to those illustrated photographically in previous reports by Newton (1976a and 1976b). The creation and eventual collapse of cavities in the deposits by induced recharge is the same process described by many authors as "piping."

In an area of sinkhole development where the cone of depression is maintained by constant pumpage (fig. 5), all mechanisms described are active even though one may be responsible for the development of a specific collapse. In contrast, a cavity resulting from a loss of support (site 3) can be enlarged and collapsed by induced recharge if it has intersected openings interconnected with the surface. Similarly, in an area near the outer margin of the cone (site 2), the creation of a cavity and its collapse can result from all mechanisms. It can originate from a loss of support, can be enlarged by the continual addition and subtraction of support and alternate wetting and drying resulting from water-level fluctuations, can be enlarged by the increased velocity of movement of water against sediment that originally filled the openings (fig. 3), and can be enlarged and collapsed by induced recharge entering from the surface.

Occurrence and Size.--Sinkhole activity due to water-level declines is confined to small areas that generally vary in size from a square meter to about 26 km^2. Water withdrawals from sinkhole active areas has generally ranged from 1,900 to 76,000 m^3 per day with the resulting water level declines generally varying from 6 to 107 m. Activity within the areas is often intense and prolonged. Near Tampa, Florida, 64 collapses reportedly occurred within a 1.6 km radius of a well field in 1964 (Sinclair, 1982). Five other sites experiencing one to thirty collapses in the same general area were also reported. Near Jamestown, South Carolina, 42 collapses occurred within a cone of depression (Spigner, 1978). In Alabama, an estimated 1,700 collapses or related features have occurred in five areas examined (Newton, 1976a). In Pennsylvania, about 100 collapses have occurred in a cone of depression near Hershey (Foose, 1953). Near Friedensvile, records indicated that 128 sinkholes formed in an area around a point of withdrawal from 1953-57 and 25 new sinkholes were recorded during a four month period ending January 1971 (Pennsylvania Department of Transportation, 1971). Sites of similar intense development, in addition to those described, were identified in Alabama, Georgia, Maryland, North Carolina, Pennsylvania, South Carolina, and Tennessee.

Data available relating to the size of sinkholes resulting from ground water withdrawals are limited. Most are described as being "small." Of 42 collapses described in South Carolina (Spigner, 1978), the largest was about 6 m in diameter, and the greatest depth exceeded 3 m. The largest of 64 collapses near Tampa, Florida was reported to have the same dimensions (Sinclair, 1982). Collapses in Alabama generally range from 1 to 100 m in diameter and from 1 to 30 m in depth. An inventory of 243 collapses in two areas (Newton and Hyde, 1972; Newton and others, 1973) showed that the average sinkhole in the first area was 3.7 m long, 3 m wide, and 2.4 m deep. The average sinkhole in the second area was about 6.1 m long, 4 m wide, and 2.1 m deep. In Shelby County, Alabama, six collapses observed had diameters approaching or exceeding 30 m. Collapses with diameters generally ranging from 7.6 to 15.2 m were not uncommon. Some near Sylacauga, Alabama, had surface dimensions of 9 to 30 m. In Hershey Valley, Pennsylvania, 100 new sinkholes were reported to be 0.3 to 6.1 m in diameter and 0.6 to 3 m deep (Foose, 1953).

Construction

The term "construction" applies to the erection of a structure, to any modification of the land surface, and to the diversion and impoundment of water. Diversion of drainage also includes any activity that results in changes in the downward movement of recharge. These activities include removal of timber and drilling, coring, and augering where pumpage is not involved. Also included is leakage from sewers, pipes, and similar facilities.

Construction practices often "set the stage" for sinkhole occurrence. Grading results, in cuts, in the thinning of unconsolidated deposits. Emplacement of weight on thinned roofs of existing cavities in residual clay or on those of shallow bedrock cavities can cause their failure. The occasional collapse beneath heavy equipment during construction is probably attributable to this cause. Differential compaction caused by the weight of a structure on unconsolidated deposits overlying the irregular surface of the top of bedrock also results in subsidence and foundation problems. Rainfall and saturation of roofs of underlying cavities in residual clay after grading can also result in their failure.

Shocks or vibrations resulting from blasting can also cause or contribute to the failure of roofs of cavities in bedrock and unconsolidated deposits. About four percent of collapses identified in Missouri have been attributed to this cause (Williams and Vineyard, 1976).

Concentration of water by drainage diversion may increase recharge to underlying bedrock openings. It also can cause saturation and weakening of roofs of existing cavities in unconsolidated deposits. Collapses due to this occurrence are less common than those caused by the creation and enlargement of openings in the deposits that result from the movement of water to and through existing openings in the top of bedrock. The subsurface erosion of unconsolidated deposits and the creation or enlargement of resulting cavities has been described by Newton (1976a) and Moore (1980). The former described it as being the same as the "piping" process resulting from induced recharge caused by a decline in the water table. The latter identified the process as being responsible for collapse failures in Tennessee.

Collapses resulting from the piping process would be most common where the water table is located below the top of bedrock (site 1 on fig. 3). The erosive energy of downward moving water dissipates when it encounters a water table located above an opening in bedrock.

Collapses resulting from leakage from underground pipes are well documented in the literature. Resulting collapse mechanisms are the piping process and saturation. Saturation causes loss of cohesion of residual clays and also causes loading due to the addition of the weight of water.

Collapse also can result where surface water gains access to uncased or unsealed holes created by drilling, augering, or coring. The piping process is generally responsible. The same process may be responsible for collapses that occur at drainage wells.

Collapses caused by the impounding of drainage occur, in part, in the same manner as those resulting from diversions of drainage. The impounding of water results in the saturation and loss of cohesiveness of unconsolidated deposits overlying bedrock openings. This, accompanied by loading resulting from the weight of impounded water, can result in the collapse of overlying deposits into the bedrock opening and a draining of the impoundment (Aley and others, 1972). If the impoundment is located where the water table is below the top of bedrock and openings at the surface are interconnected with those in bedrock, a collapse can result from the piping process. Collapses resulting from saturation and piping have been described by Warren (1974).

The piping process can also result in sinkholes when water is impounded on unconsolidated deposits where the water table was originally located above the top of bedrock. On the floor of the impoundment, water moving under increased head through openings in the deposits into openings in underlying rocks can both form and cause collapse of cavities in the deposits. This would occur where there is considerable pressure exerted by the impounded water and where openings in underlying carbonate rocks have a discharge point outside of the impoundment at a lower altitude. The increase in the velocity of movement of water through openings in unconsolidated deposits into underlying openings in bedrock, resulting from the pressure, would probably have an erosive capacity comparable to that in a cone of depression caused by pumpage. This action is probably responsible for the formation of some sinkholes in a large impoundment on the Coosa River in Alabama (Newton, 1976a).

<u>Occurrence and Size</u>.--Sinkholes resulting from diversions of drainage have occurred in most States in which active subsidence has been reported.

Numerous sites attributable to this cause have been inventoried in Alabama and Georgia and many of the more than 200 sinkholes reported by the Florida Department of Transportation (written commun., 1981) as having occurred in and near their right-ofways are probably attributable to this cause. Collapses associated with highway construction in Missouri are due to changes in the water regimen (Williams and Vineyard, 1976). Collapses attributable to this cause along highways in Tennessee have been described and illustrated (Moore, 1980). Similarly, collapses beneath drains in Pennsylvania and Kentucky are attributed to this cause. About 20 collapses resulting from concentration of drainage occurred at one site in Pennsylvania (Knight, 1970).

Sinkholes resulting from the impounding of water have occurred in numerous States. Impoundments affected extend from Florida in the south to Minnesota in the north. Some of the larger impoundments are located in Alabama, Arkansas, Georgia, Kentucky, Missouri, and North Carolina.

Most collapses due to construction are relatively small. Most in Alabama are less than 5 m in diameter. The largest surface area involved was about 18 m in diameter and the greatest depth about 8 m. These dimensions are similar to the largest reported along roadways in Florida (Florida Department of Transportation, written commun., 1981). The largest reported in Missouri was about 23 m long, 11 m wide, and 8 to 9 m deep (Aley and others, 1972).

NATURAL SINKHOLES

Many thousands of sinkholes dot landscapes of carbonate terranes in the eastern United States. Of those shown on topographic maps, almost none represent the earliest stage of their development.

The evolution of a natural sinkhole occurs in a geologic time frame. The enormity of the time span involved, in comparison to that involved in induced sinkhole development, should be considered in descriptions that follow. The rate of solution, for instance, should be considered where progressive solution of bedrock removes its upper surface and enlarges openings or caverns in it. A summary of estimates by previous workers as described by Sweeting (1973) indicates rates of solution for most terranes (lowering of land surface) that are less than 100 mm per 1,000 years.

In the selected terrane, most natural sinkhole activity is restricted to the highland area (fig. 3). Their occurrence in lowlands is comparatively rare because openings there have been subjected to solution for a shorter period of time and because deposition rates tend to maintain a level surface over subsiding areas.

Progressive Solution of Bedrock and Decline of Water Table

The development of a new natural sinkhole may reflect displacement of bedrock, the unconsolidated deposits overlying it, or both. The displacement of either or both is generally "triggered" by progressive solution of bedrock, by a natural decline in the water table, or by a combination of both.

The role of solution in sinkhole development is recognized by all investigators. The effect that solution of bedrock or a decline in the water table has on unconsolidated deposits regarding development of natural sinkholes is not nearly as well defined. Previous investigations have associated the water table and some forces resulting from natural declines with the development of sinkholes. Herrick and LeGrand (1964) related solution subsidence to changes in base level and also related recent sinkhole activity along the Flint River in southwest Georgia to the entrenchment of the stream that resulted in a lowering of base level. Sweeting (1968) listed, among other factors controlling the development of sinkholes, the "variations in the water level within the limestones; violent and rapid fluctuations in the water level as occur in some tropical humid areas." Where the zone of saturation is high enough to fill caverns near the land surface, a lowering of the water level removing some support to the roof may trigger a collapse (Springfield and LeGrand, 1969).

Displacement of unconsolidated deposits overlying openings in bedrock is the most common mode of natural sinkhole development. Cavities in unconsolidated deposits that have or could eventually result in natural sinkholes have been described by many investigators including Newton (1976a) and Williams and Vineyard (1976).

The downward migration of unconsolidated deposits due to natural declines in the water table accompanying a lowering of base level is considered here to be an integral part of the solution process. Major differences between the formation of natural sinkholes resulting from collapses in bedrock and collapses in unconsolidated deposits include the time required for each to develop. The time required for a cavity to form in unconsolidated deposits due to change in the hydrologic regimen would be extremely short when compared to that required for the enlargement by solution of a cavity in bedrock to the point where its roof becomes incompetent.

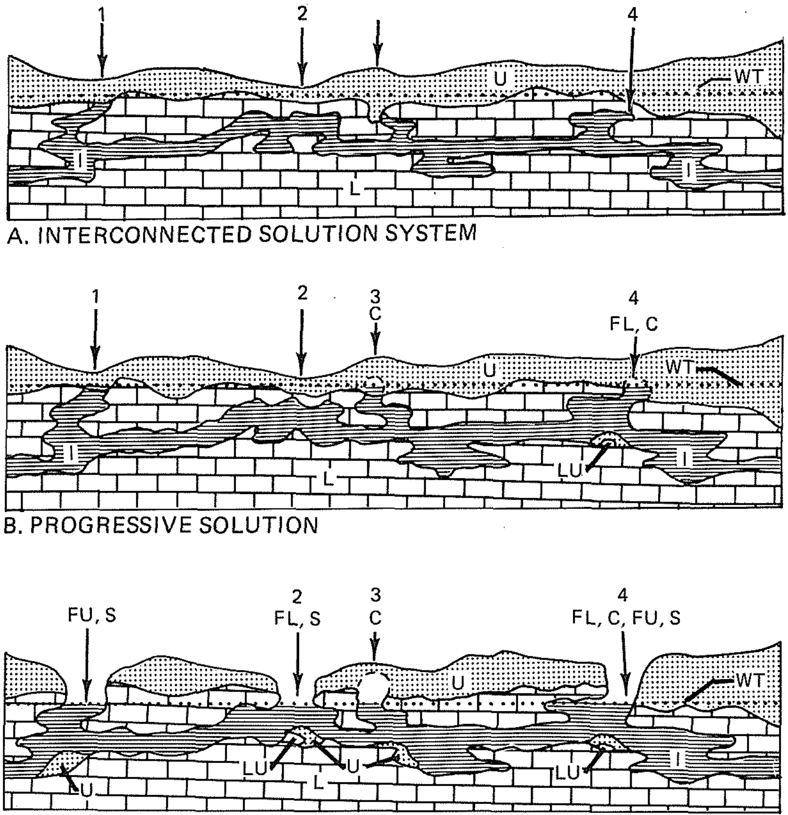
Subsidence resulting from solution of the top of bedrock or openings in it may be the most common mode of sinkhole development. This occurrence in a geologic time frame results from the downward adjustment of soil and other overburden as the underlying bedrock is removed by solution. Collapse, as a form of adjustment, would not play a role in the initial stage of development. This mode of development may also account for the rarity of collapses reported in some terranes containing numerous older sinkholes.

Natural declines in the water table are caused by the entrenchment of streams or by the solutional enlargement of bedrock openings. The declines trigger, in a geologic time frame, the same mechanisms or mechanical processes that result in induced sinkholes (Newton, 1976a). Progressive solution of bedrock, the resulting decline in the water table, and the development of sinkholes in the highland area in the selected terrane (fig. 3) is illustrated schematically in figure 6. The water level decline (fig. 6C) resulted when openings became large enough to store and transmit more water to their point of discharge than the amount received from recharge.

Solutional enlargement of an opening in bedrock results in a thinning of its roof. Failure occurs when the roof can no longer support its weight and the weight of overlying deposits. The collapse results in the development of a sinkhole at the surface or in a cavity in overburden caused by the downward migration of unconsolidated deposits into the bedrock opening created. The thinning of a bedrock roof by solution is illustrated at site 2 in figure 6A and 6B. Its failure in figure 6C could have resulted from additional solution or loss of support caused by the water level decline. A similar roof failure and the creation of an overlying cavity in unconsolidated deposits is shown at site 4 in figure 6A and 6B. Water-level fluctuations against the roof of the cavity would result in its later enlargement and collapse (fig. 6C). Natural collapses of similar cavities in Montgomery County, Tennessee have been attributed to this mechanism (Kemmerly, 1980).

Solutional enlargement of an opening in the top of bedrock at site 1 (fig. 6A and 6B) and a loss of support due to the water-level decline (fig. 6C) resulted in the collapse of the unconsolidated deposits spanning the opening. A similar occurrence at site 3 resulted in the downward migration of unconsolidated deposits and formation of a cavity in the overburden. Prior to a decline in the water table (fig. 6C), this cavity was enlarged by waterlevel fluctuations.

The piping process plays a significant role in natural sinkhole development. Prior to the natural decline in the water table (fig. 6C), it could



A. INTERCONNECTED SOLUTION SYSTEM

B. PROGRESSIVE SOLUTION

C. PROGRESSIVE SOLUTION AND DECLINE OF WATER TABLE

EXPLANATION

U - UNCONSOLIDATED DEPOSITS
FU - FAILURE OF UNCONSOLIDATED ROOF
I - INTERCONNECTED SOLUTION SYSTEM
C - CAVITY IN UNCONSOLIDATED DEPOSITS
LU - LIMESTONE FRAGMENTS AND UNCONSOLIDATED DEPOSITS
FL - FAILURE OF LIMESTONE ROOF
L - LIMESTONE
WT - WATER TABLE
S - SINKHOLE

(Numbers apply to sites described in report.)

Figure 6. Natural sinkhole development.

contribute only to the enlargement and collapse of cavities in unconsolidated deposits that extended above the water table (sites 3 and 4 on fig. 6B). Elsewhere, the erosive energy of recharge moving from the surface to an underlying opening dissipated when it encountered the water table. After the decline (fig. 6C), piping would probably be the major mechanism causing sinkholes.

Drought and Decline of Water Table

The only natural decline in the water table associated with sinkhole activity generally observable by man occurs during drought, an extended period of less than average precipitation. Although drought and accompanying waterlevel declines are probably one of the most recognized or accepted causes of sinkhole development, mechanisms triggering sinkholes have received little or no attention.

During a drought, openings in bedrock receive less recharge. They continue to discharge; however, into nearby streams. This discharge, causing a loss of storage, results in a decline in the water table and the drying up of springs and in declines in stream level. Some streams go dry.

The decline in water level to a position below the top of bedrock triggers the same mechanisms, with the exception of an increase in velocity, that cause sinkholes resulting from induced declines. A loss of support occurs and water-level fluctuations resulting from the less than average rainfall move though openings in the rock against the base of overlying unconsolidated deposits. Recharge from the less than average rainfall, with the water level below the top of bedrock, also results in collapses due to the "piping process.", In contrast to the increase in velocity occurring in a cone of depression during pumpage, the velocity under these conditions decreases due to a flattening of the hydraulic gradient.

Occurrence and Size

New natural sinkholes were identified in most States underlain by carbonate rocks. The most active areas, based on available information are Alabama, Florida, Georgia, Missouri, Pennsylvania, and Tennessee. How significant sinkhole development is in most areas is unknown because of the scarcity of investigations dealing with their occurrence. The number of natural collapses in Florida exceeds that of any of the other States. The actual number may not be as large as that conceived by many; however, because of the limited investigations made there.

Many natural sinkholes occur, like induced sinkholes resulting from water-level declines, during periods of greater than average precipitation. In Missouri, for example, 13 catastrophic collapses occurred in remote timbered areas during a 2-month period in 1973 when precipitation was 3 to 6 times higher than normal (Williams and Vineyard, 1976). Similarly, 12 collapses occurring during a 3-month period of greater than average precipitation in Montgomery County, Tennessee in 1975 contrasted with 6 collapses occurring during the preceding 3 months when precipitation was less than average (Kemmerly, 1980).

Sinkhole activity in several areas in Alabama occurred during a prolonged drought in the early and middle 1950's. More than 40 collapses that occurred in Sylacauga, Alabama were attributed to a decline in the water table during the drought (George Swindel, written commun., 1971). Groundwater withdrawals were undoubtedly a contributing factor during this period; however, a return of normal precipitation and recovery of the water table was followed by the cessation of or drastic decrease in sinkhole activity. A similar occurrence at Murphresboro, Tennessee happened during a drought in the 1940's. Many sinkholes occurring in Florida in 1981 have also been attributed to drought conditions.

Surface dimensions of collapses forming natural sinkholes in the study area generally range from 1 to 105 m. Depths generally vary from less than 1 to more than 30 m. Most reported had maximum surface dimensions and depths that were considerably smaller than 6 m. The largest in Alabama was estimated to be 30 m long, 23 m wide, and 9 m deep (G. Swindel, oral commun., 1974). Thirteen collapses in remote areas in Missouri in 1973 were 6 to 8 m in diameter and 11 to 18.3 m deep (Williams and Vineyard, 1976). Surface dimensions of 18 collapses in Montgomery County, Tennessee ranged in length from 10 to 64 m and in width from 7 to 43 m (Kemmerly, 1980). Based on information for 225 natural and induced sinkholes forming along or near highways in Florida, diameters generally ranged from 1 to 68 m and depths from 1 to 46 m (Florida Department of Transportation, written commun., 1981).

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A SINKHOLE NEAR THE DAMASCUS GATE, OLD CITY OF JERUSALEM

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Abstract

The area in the vicinity of the walls of the Old City of Jerusalem has been reshaped, several times, during the past centuries. Buildings have been destroyed and areas have been covered or filled-in, with rubble and building waste material. Therefore sites of national, ethnical or archaeological significance may be found in present day construction projects and may present a hazard to construction. This repeated activity has artificially changed the topography, left "underground" openings and has altered the natural drainage regime of the area. As a consequence of unusually high and concentrated rainfall, sinkholes of various sizes appeared in this ground. The present paper deals with the use of old maps, geophysics and borings to determine the configuration of the natural topography, type of fill material and depth to bedrock. On the basis of this data a mechanism of sinkhole development is proposed in order to evaluate suitable remedial measures required in areas where construction is planned or taking place.

General

On the 7th January 1980 a sinkhole developed in the ground, in a flat area west of the Damascus Gate, Old City of Jerusalem (Figs. 1,5). Subsidence occurred at the development site of a major road junction, forming a sinkhole of 8m diameter and over 6m deep (Fig 2) with associated concentric fractures (Fig 3) and smaller sinkholes (Fig 4) up to 5m away from the main subsidence.

This type of phenomena, although considered to be an uncommon hazard in the prevailing climate of Jerusalem, caused grave concern with regard to the stability of the area in general. Particulary since the winter of 1979-80 appeared to cause an "epidemic" of sinkholes in various parts of the country after unusually high and concentrated rainfall.

A detailed geotechnical survey of the area in the vicinity of the Damascus Gate was carried out to determine the type and depth of fill material, the presence of karstic openings in the limestone bedrock and man-made openings such as buried cellars, water wells and aquiducts. Each one of these types of openings was considered to be a reasonable initiator of the development of a sinkhole.

The Site

An area of approximately 200 x 250m located at the road junction west of the old city walls near the Damascus Gate (Figs. 1,5) was examined. The area is situated at the eastern end of a present day wide flat valley filled with natural and man-made materials. The valley drains the north-western suburbs of Jerusalem (Morasha, Russian Compound, Mea She'arim, Romema) towards the old city. The present day surface and topography does not conform to the natural configuration of the valley or the natural water course. Today's surface slopes to the north-west whereas the bedrock slopes south-east towards the old city walls.

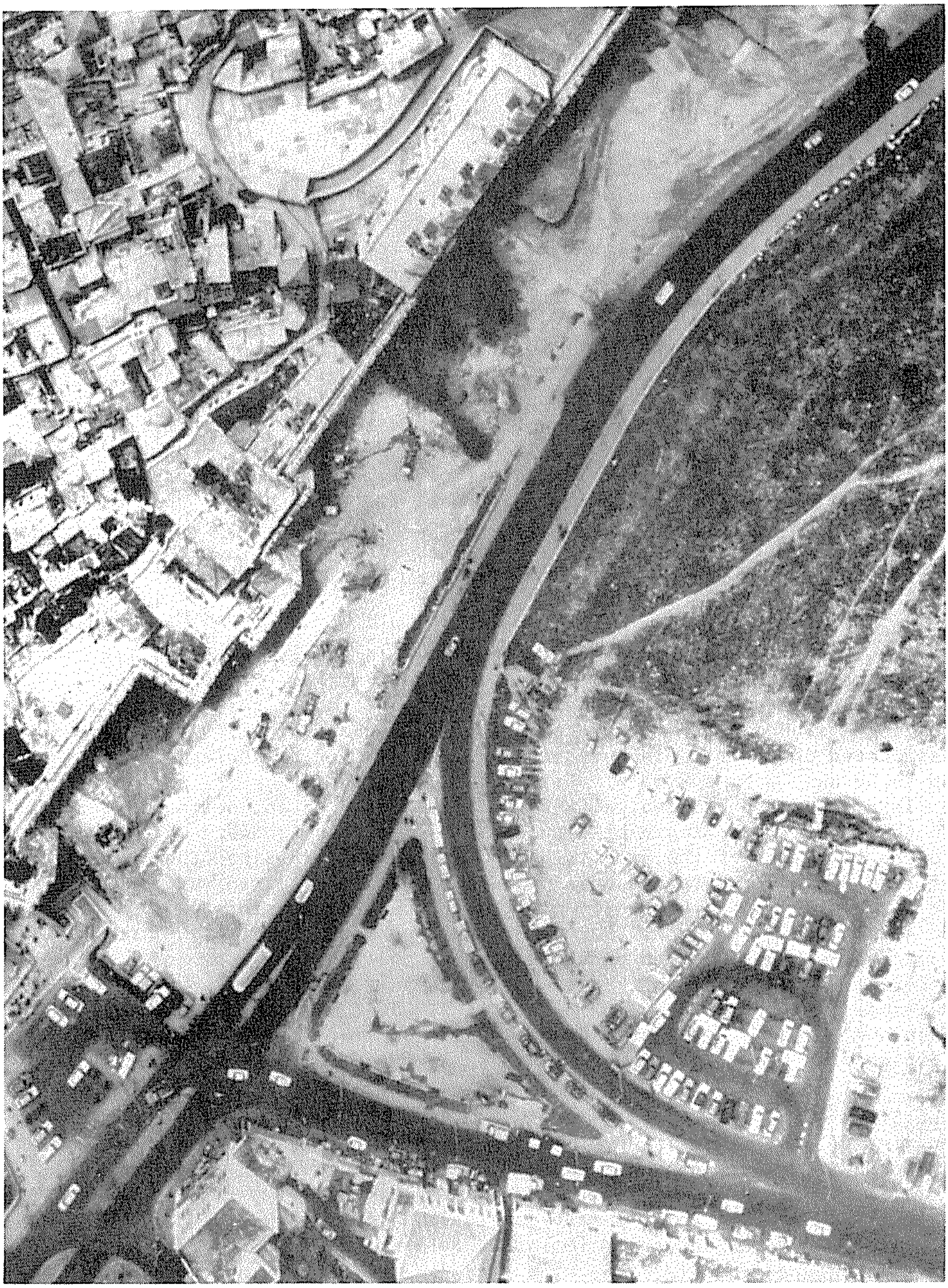


Fig.1: Aerial view of the Damascus Gate and road junction before recostruction. Photographed 1978.

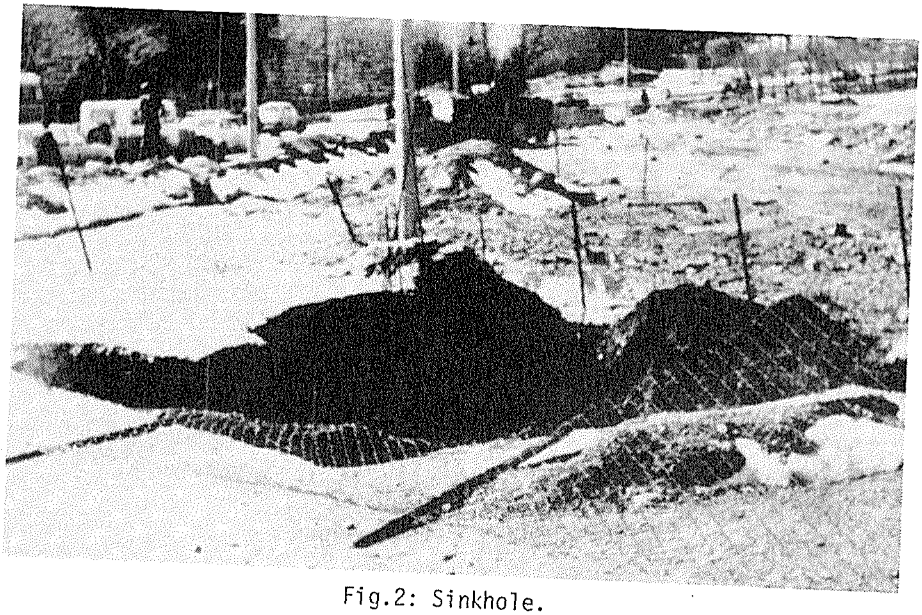


Fig.2: Sinkhole.

Fig.3: Surface fracture.

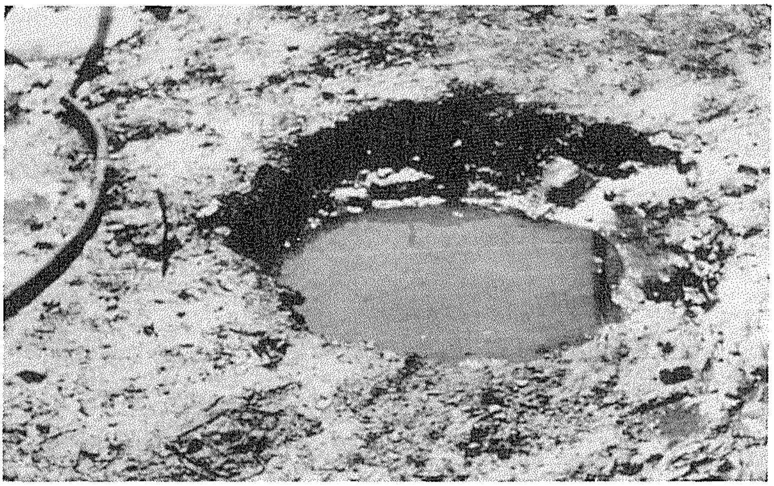


Fig.4: Secondary small sinkhole.

The bedrock and the surrounding hills consist of well bedded hard, dense limestone of the Bina Fm., of Turonian Age (Arkin et. al. 1973) and is a common building stone in Jerusalem and its vicinity. The rock is a pale yellow, micritic limestone, iron stained, cut by calcite veins and contains fossil fragments. Solution features in the form of karstic caves of several cubic meters volume and associated with faults and major fractures are common. Smaller solution holes of several centimeters across are also common throughout the formation.

The Damascus Gate sinkhole developed following several rainstorms of up to 90mm rainfall spread out over the winter months of December 1979 and January 1980. The repeated rainstorms, at intervals of 3-5 days did not allow the drying out of the ground as normally occurred during previous years.

The Survey

An examination of old maps (Figs. 6,7) of the area (Wilson, 1864; Deutschen Veren Zur Eroforschang Platestinas, 1904) and modern topographic maps (Fig. 5) showed that many changes in topography are evident, particularly the position of the deepest part of the valley floor. The maps also revealed the location of aquiducts, water wells and old buildings (cellars) now covered and hidden.

As a first approach to the problem a detailed base map was prepared on a scale of 1:5,000 including present day construction data and data from the old maps.

A comparison of the above maps enabled the planning and execution of a geophysical refraction survey (Fig. 8) so that the lines chosen would provide maximum data on the thickness of the valley fill, depth to rockhead and verification of possible buried opennings. The survey provided the position and alignment of the main ancient water course and showed that the sinkhole had developed on the northern bank. The depth from the surface to rockhead in the center of the valley reached 13.5m and on the banks, up to 9m. A second (tributary) water course was revealed as flowing from south-west to north-east parallel to the old city wall and joining the main course near the Damascus Gate.

Twelve auger and core drillings were carried out to verify the type of fill material and thickness. Figures 9,10,11 show examples of the material

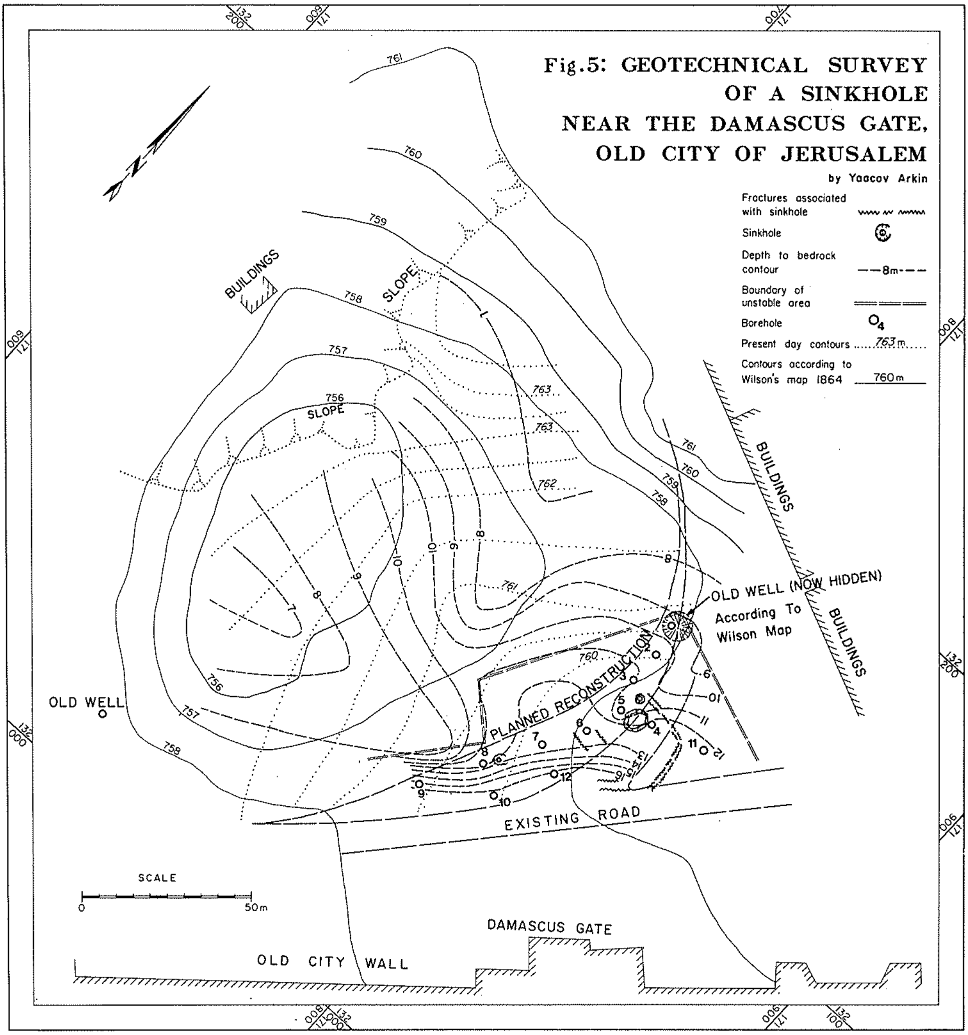


Fig.5: GEOTECHNICAL SURVEY OF A SINKHOLE NEAR THE DAMASCUS GATE, OLD CITY OF JERUSALEM
by Yaacov Arkin
Fractures associated with sinkhole
Sinkhole
Depth to bedrock contour
8m
Boundary of unstable area
Borehole
Present day contours
763 m
Contours according to Wilson's map 1864
760 m
BUILDINGS
SLOPE
OLD WELL
OLD WELL (NOW HIDDEN) According To Wilson Map
PLANNED RECONSTRUCTION
EXISTING ROAD
SCALE
0
50m
DAMASCUS GATE
OLD CITY WALL
761
760
759
758
757
756
763
762
132 200
171 600
171 700
171 800
132 000
171 900
132 100

Fig. 5

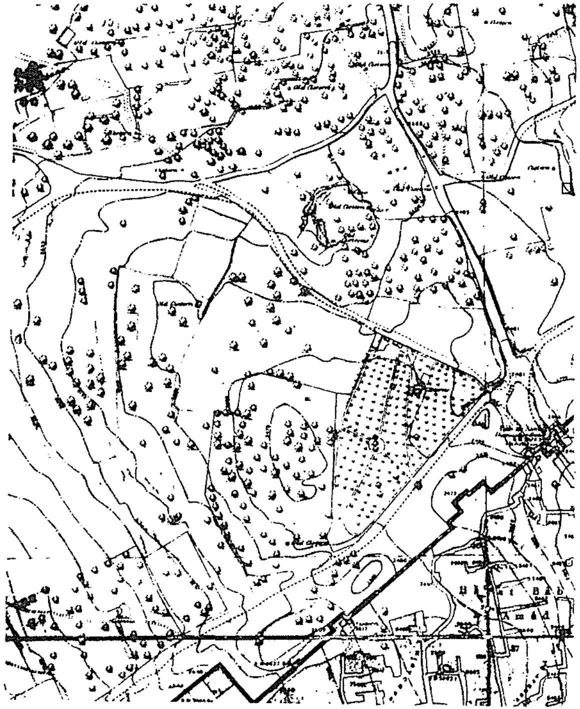


Fig.6: Ordinance Survey of Jerusalem.
by Captain Charles W.Wilson R.E., under the
direction of Colonel Sir Henry James R.E.F.
R.S.C. Director of Ordinance Survey, 1864-5.

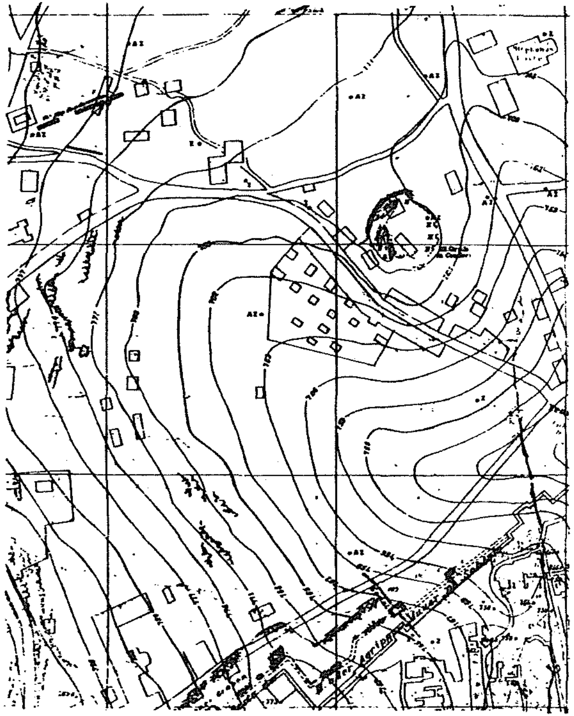


Fig.7: Karte Der Materialien Zur Topographie Des
Jerusalem Gezeichnet von Oberleher August
Kummel Barmen,und Herausgegehen Vom.
Deutschen Veren. Zur Erforschung
Palestinas 1904.

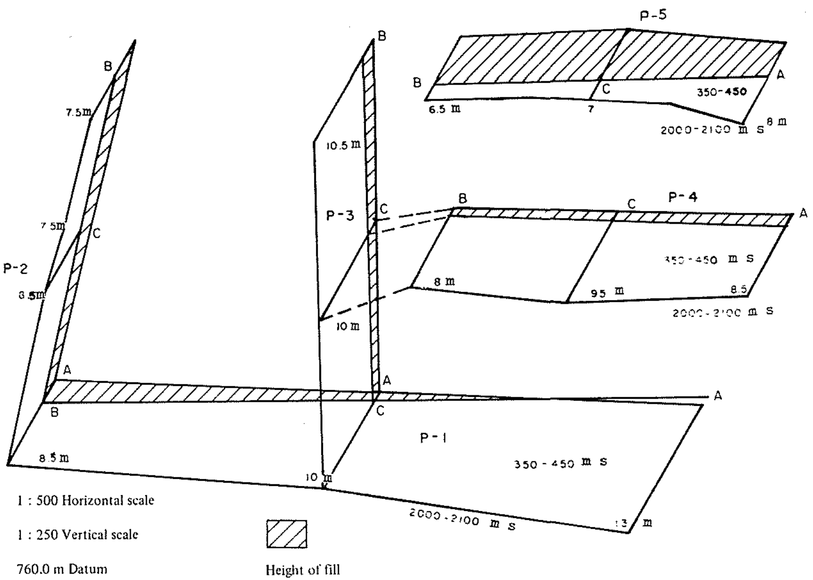
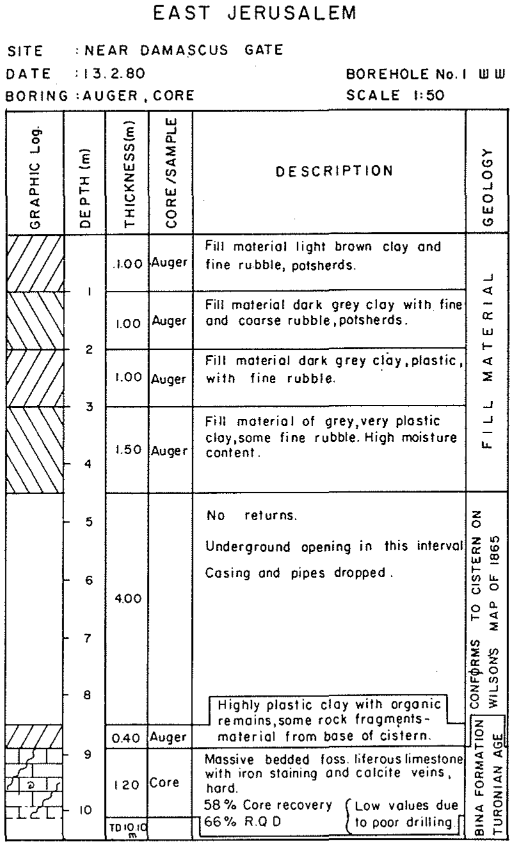


Fig. 8: GEOPHYSICAL REFRACTION SURVEY FOR DETERMINING DEPTH OF FILL MATERIAL

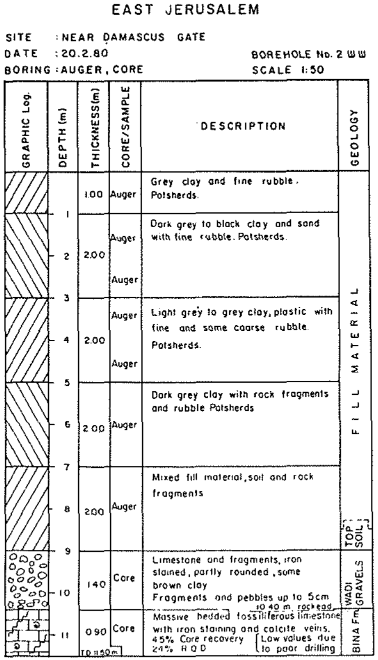


EAST JERUSALEM

SITE : NEAR DAMASCUS GATE
DATE : 13.2.80
BORING : AUGER, CORE
BOREHOLE No. 1 ɯ ɯ
SCALE 1:50

GRAPHIC Log.	DEPTH (m)	THICKNESS(m)	CORE/SAMPLE	DESCRIPTION	GEOLOGY
		1.00	Auger	Fill material light brown clay and fine rubble, potsherds.	FILL MATERIAL
		1.00	Auger	Fill material dark grey clay with fine and coarse rubble, potsherds.	
		1.00	Auger	Fill material dark grey clay, plastic, with fine rubble.	
		1.50	Auger	Fill material of grey, very plastic clay, some fine rubble. High moisture content.	
		4.00		No returns. Underground opening in this interval Casing and pipes dropped.	CONFORMS TO CISTERN ON WILSON'S MAP OF 1865
		0.40	Auger	Highly plastic clay with organic remains, some rock fragments-material from base of cistern.	
		1 20	Core	Massive bedded fossiliferous limestone with iron staining and calcite veins, hard. 58 % Core recovery } Low values due 66 % R.Q D } to poor drilling	BINA FORMATION TURONIAN AGE
		TD 10.10 m			

Fig. 9: SUBSIDENCE-NEVI'IM-ZANHANIM JUNCTION



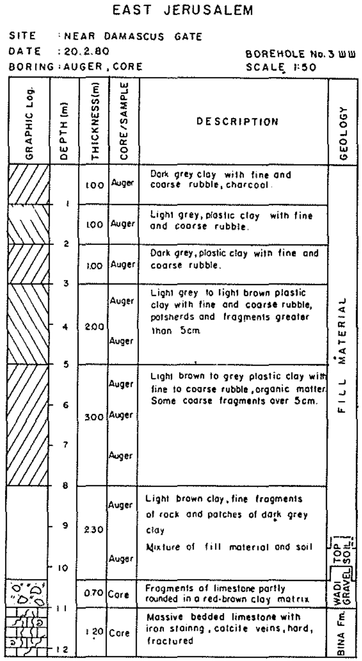
EAST JERUSALEM

SITE : NEAR DAMASCUS GATE
DATE : 20.2.80
BORING : AUGER, CORE

BOREHOLE No. 2 WW
SCALE 1:50

DEPTH (m)	THICKNESS(m)	CORE/SAMPLE	DESCRIPTION	GEOLOGY
1	1.00	Auger	Grey clay and fine rubble. Potsherds.	FILL MATERIAL
2 3	2.00	Auger Auger	Dark grey to black clay and sand with fine rubble. Potsherds.	
4 5	2.00	Auger Auger	Light grey to grey clay, plastic with fine and some coarse rubble. Potsherds.	
6 7	2.00	Auger	Dark grey clay with rock fragments and rubble Potsherds	
8 9	2.00	Auger	Mixed fill material, soil and rock fragments	TOP SOIL
10	1.40	Core	Limestone and fragments, iron stained, partly rounded, some brown clay Fragments and pebbles up to 5cm 10.40 m. rockead	WADI GRAVELS
11	0.90 TD 11.50m	Core	Massive bedded fossiliferous limestone with iron staining and calcite veins. 45% Core recovery 24% R Q D Low values due to poor drilling	BINA Fm.

Fig. 10: SUBSIDENCE-NEVI'IM-ZANHANIM JUNCTION



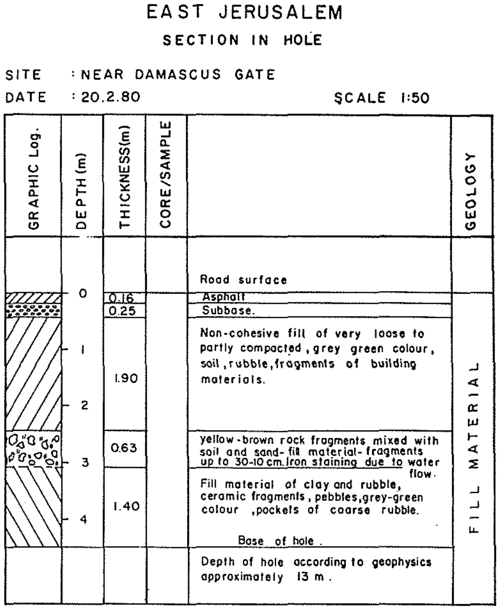
EAST JERUSALEM

SITE : NEAR DAMASCUS GATE
DATE : 20.2.80
BORING : AUGER, CORE

BOREHOLE No. 3 WW
SCALE 1:50

DEPTH (m)	THICKNESS(m)	CORE/SAMPLE	DESCRIPTION	GEOLOGY
1	1.00	Auger	Dark grey clay with fine and coarse rubble, charcool.	FILL MATERIAL
2	1.00	Auger	Light grey, plastic clay with fine and coarse rubble.	
3	1.00	Auger	Dark grey, plastic clay with fine and coarse rubble.	
4 5	2.00	Auger Auger	Light grey to light brown plastic clay with fine and coarse rubble, potsherds and fragments greater than 5cm.	
6 7 8	3.00	Auger Auger Auger	Light brown to grey plastic clay with fine to coarse rubble, organic matter. Some coarse fragments over 5cm.	
9 10	2.30	Auger Auger	Light brown clay, fine fragments of rock and patches of dark grey clay Mixture of fill material and soil	TOP SOIL
11	0.70	Core	Fragments of limestone partly rounded in a red-brown clay matrix	WADI GRAVEL
12	1.20	Core	Massive bedded limestone with iron stainng, calcite veins, hard, fractured	BINA Fm.

Fig. 11: SUBSIDENCE-NEVI'IM-ZANHANIM JUNCTION



EAST JERUSALEM

SECTION IN HOLE

SITE : NEAR DAMASCUS GATE
DATE : 20.2.80

SCALE 1:50

DEPTH (m)	THICKNESS(m)	CORE/SAMPLE		GEOLOGY
			Road surface	
0	0.16		Asphalt	FILL MATERIAL
	0.25		Subbase.	
1 2	1.90		Non-cohesive fill of very loose to partly compacted, grey green colour, soil, rubble, fragments of building materials.	
3	0.63		yellow-brown rock fragments mixed with soil and sand-fill material-fragments up to 30-10 cm. Iron staining due to water flow.	
4	1.40		Fill material of clay and rubble, ceramic fragments, pebbles, grey-green colour, pockets of coarse rubble. Base of hole.	
			Depth of hole according to geophysics approximately 13 m.	

Fig. 12: SUBSIDENCE-NEVI'IM-ZANHANIM JUNCTION

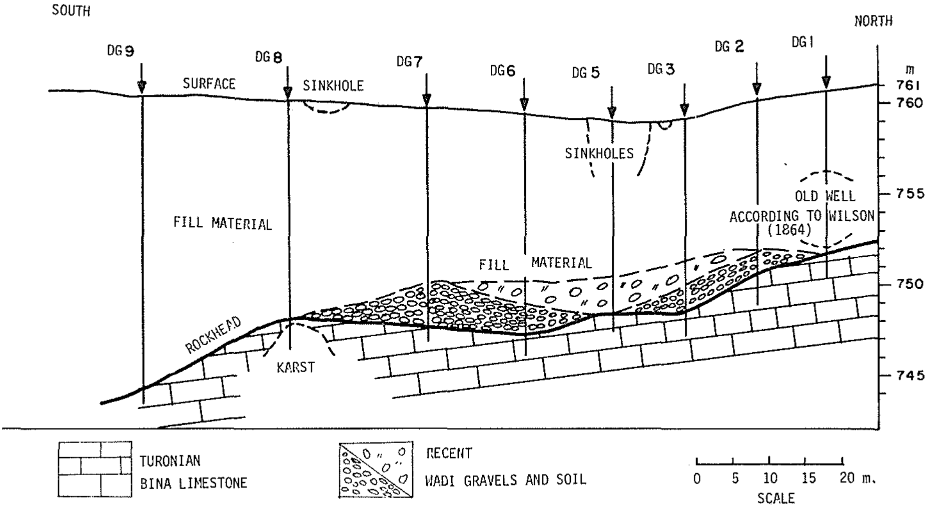


Fig. 13: CROSS SECTION JOINING BOREHOLES 1-9.

penetrated and its characteristics. These were compared to the section exposed in the wall of the sinkhole (Fig. 12). The fill material consists of water sensitive clay, building waste of blocks up to 30-50cm across mixed with fine and powdery material. A well developed bedding and grading of the different size materials is observed and the range in thickness of the beds, of up to 1.50m was formed due to the manner in which the material was thrown down. The coarse material forms well defined beds of extremely high permeability that act as water conduits.

Bedrock in the center of the valley and along the ancient water course is covered by a gravel layer up to 1.50m thick (Figs. 10,11). The gravels consist of well rounded pebbles 5-10 cms across, unconsolidated and non-cemmented, forming a highly permeable layer.

A number of karstic holes of several meters depth and cisterns (Fig. 9) were penetrated in the borings. These were found to be filled in most cases, by terra rosa soil in the former and organic clays in the latter.

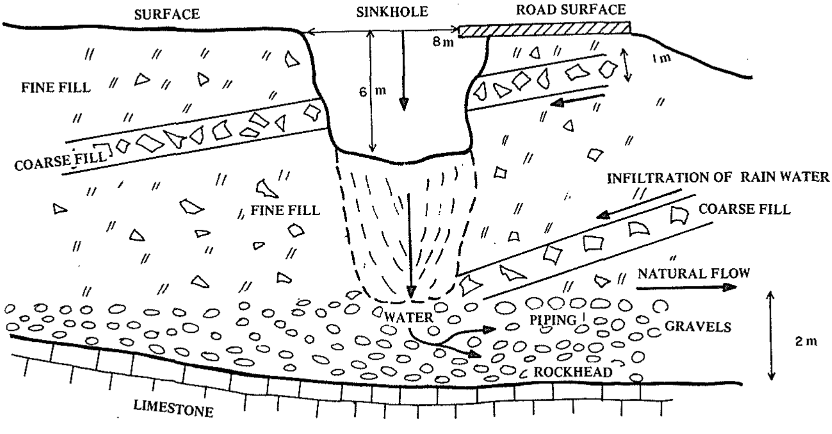


Fig. 14: MECHANISM OF SINKHOLE DEVELOPMENT

A cross-section (Fig. 13) joining borehole 1-9 shows the relationship between the sinkholes, fill material, wadi gravels and rockhead.

Mechanism of Subsidence

Subsidence began after a wet period in which 90mm of rainfall fell in cycles of 2-3 days of rain and 3-5 days of dry weather. At this rate of rainfall the subsurface did not have sufficient time to drain and the various clay layers became saturated. The coarse fill material acted as highly permeable layers distributing the water throughout the fill.

The sinkhole developed at the surface above the intersection of permeable fill layers and the gravel bed in the ancient water course, due to the opposing dips of these units. This situation allowed the water content to increase greatly and rapidly at the intersection and adjacent to it. The subsequent rise in pore pressure and loss in strength of the clays resulted in piping and suction of the fine material into the porous gravels. This led to subsidence at the surface and the formation of the sinkhole. Several smaller sinkholes up to 2m diameter were formed after further rains. Figure 14 shows the relationship between the different layers and the direction of flow.

Remedial Measures

On the basis of the above data it was possible to deleniate the unstable area where further subsidence could affect construction works (Fig. 5). Because of the highly permeable nature of the fill and its variability in size fractions, excavation and recompaction was considered to be the most suitable remedy. The aim was to reduce infiltration and permeability at and around the road junction construction site, particularly over the ancient water course. Since these measures were taken several wet winters of comparable rainfall have passed and the ground at the site has remained stable.

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Karte Der Materialien Zur Topographic Dès Alten Jerusalem Gezeichnet von Oberlehrer August Kummel Barmen, und Herausgegehen Vom. Deutschen Verein Zur Erforschung Palestinas 1904.

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SUBSIDENCE AND COLLAPSE FEATURES IN KARSTIC COASTAL PLAINS

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Abstract

Subsidence within the Florida coastal plains falls into two categories: 1) the accelerated consolidation of clays and organic soils due to dewatering; and 2) the solution of carbonate rocks or shell, producing either gradual subsidence or cavities which may form sinkholes. Case histories involving consolidation of a buried organic deposit and subsidence in sediments overlying solution features in limestone are presented as examples of different subsidence conditions.

The location of high risk areas for cavities and cavity collapses can be estimated using the intersection of lineaments formed by fracture traces and lineated depressions (dolines). The results of one study indicated that the majority of newly formed sinkholes occurred near the intersection of lineaments interpreted from aerial photography.

Introduction

Foundation problems in the karstic coastal plains of Florida usually involve consolidation of clays and organic soils, expansive or swelling clays, and the remnants of subsurface features resulting from solution of limestone. Foundations that penetrate into rock (piles) or bridge across variable subsurface foundation conditions (mat foundations) are usually less susceptible than spread footings to subsidence induced by shallow subsurface soils and cavities.

In the Gainesville, Florida region, there exists areas with expansive and non-expansive clays located near ground surface. During periods of drought, dwellings located on these clays often sustain differential settlement sufficient to produce substantial cracks in concrete and masonry. Settlement is generally related to high degree of transpiration from trees within about 12 meters of the structure. Corrective action includes tree removal and/or the use of a watering system to maintain the moisture content.

Occasionally, construction activity involving dewatering the excavation has resulted in very rapid consolidation of weak organic soils. The effect of dewatering depends upon permeability, groundwater conditions, stratigraphy, and location of the well points. In one case, a water treatment plant was being constructed next to a one story office building and parking lot. A muck-filled old stream channel extended across the construction site past an office building. Well points installed below the organic layer resulted in dewatering underlying sands. Substantial consolidation of the muck occurred resulting in foundation settlement and cracking of walls of the concrete block office building.

The collapse of roofs over cavities in soluble carbonate rocks or the piping failure of cavities extending upward into the surficial soils can produce rapid and extremely large localized subsidence. Although this is typical of sinkholes in karst terrain, occasionally the subsidence is relatively minor but sufficient to damage structures. An example of a minor subsidence on an interstate highway is discussed in an ensuing segment of this paper. Also, the results of a remote sensing study are presented which correlate the intersection of lineaments with recently developed sinkholes.

Subsidence Induced by Dewatering

Regional subsidence due to the withdrawal of water or crude oil has been observed in numerous locations throughout the world. In Mexico City, wells lowered the water table and produced accelerated consolidation of the lake bed sediments. The discovery and pumping of crude oil at Long Beach, California resulted in regional subsidence in excess of three meters until stabilized by water injection. Although these are classic examples of land subsidence, localized subsidence may occur when utilizing dewatering methods for excavations below the water table.

In Jacksonville, Florida, the site for construction of a water treatment plant had been investigated using soil borings to bedrock. Only one boring within the confines of the excavation indicated any organic materials. Except for this discontinuity, all borings showed medium to dense sands overlying the limestone bedrock. Two of the borings outside the limits of excavation also indicated a trace of muck. Figure 1 illustrates the relationship of the excavation to surrounding buildings and the approximate location of the muck deposit. The fact that muck within the proposed excavation was thin and only encountered at one boring location apparently did not warrant any concern of the engineer or contractor.

During excavation, the contractor installed a well point system to dewater a portion of the site where the depth of excavation was greater. As excavation progressed the muck deposit was exposed, resulting in a progressive flow of muck into the excavation. Sheet piling was installed (see Figure 1) to eliminate this problem and to allow excavation to proceed.

Cracks in the single story concrete block office building were observed sometime during the dewatering and excavation of the construction site. The cracks progressed until two and three diagonal cracks were formed at several corners of the office building. The width of the cracks were variable but were greater than three centimeters in width. The structure was underpinned prior to the development of additional cracks and structural distress. The parking lot and adjacent chain link fence visibly showed substantial subsidence in the area adjacent to the office building.

The primary cause of settlement was attributed to the drawdown of the water table because it appeared that the muck flow into the excavation was localized and occurred early in the dewatering operation. The close proximity of the well points to the office building (about 120 m), the high permeability of the sand, and relatively shallow depth (3 m) of the organic layer contributed to the severity of the subsidence. Although not verified, it is believed that the sands both over and under the muck layer were dewatered, which increased effective stresses and provided for rapid dissipation of pore pressure in the muck.

This case history suggests that extreme care should be taken in the performance and interpretation of information from the foundation investigations. In this case, injection grouting could have been used to establish a cutoff wall outside the excavation to reduce the inflow of water to the well points and to minimize the potential for settlement of structures located over the muck deposit.

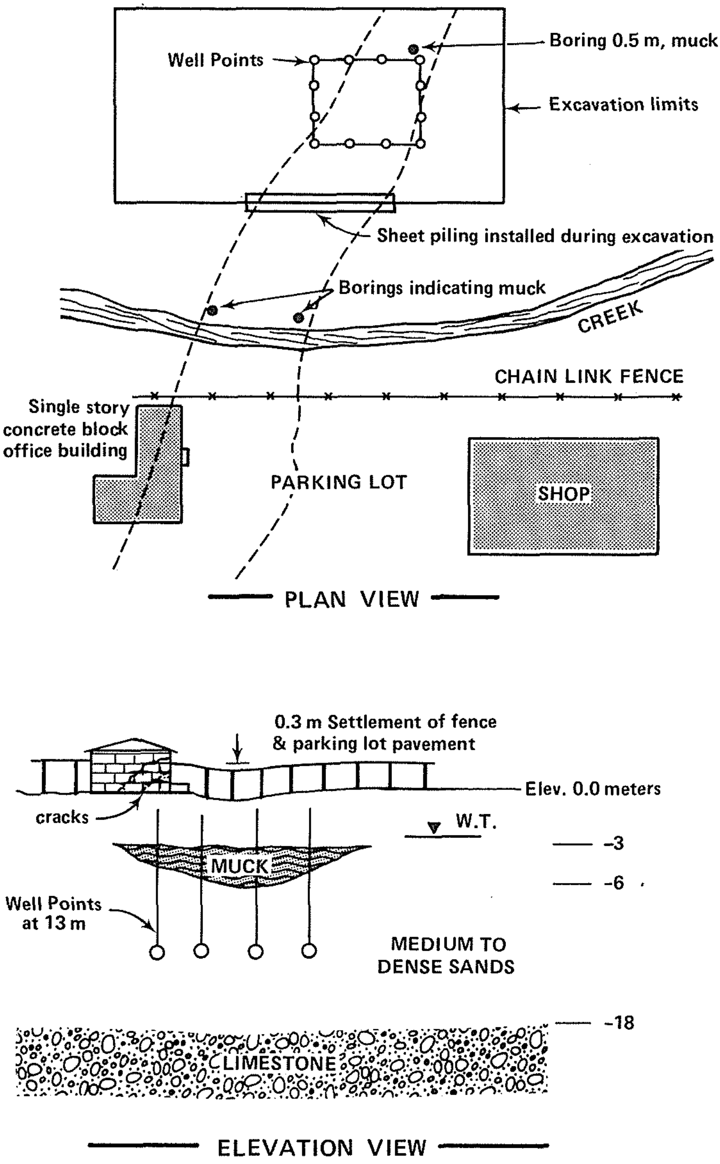


Fig. 1 Site Conditions Leading to Subsidence During Dewatering Operations

Subsidence and Collapse Characteristics in Karst Terrain

Certain regions of Florida are highly susceptible to sinkhole formation due to cavity collapse. Natural conditions, such as prolonged drought and rainfall, can assist in initiating a collapse.

Man's activities often contribute to triggering the collapse. Water table drawdown by pumping of wells for water supply, irrigation, and freeze protection of crops is a primary factor which often relates to the frequency and time of occurrence. Slight depressions and lineated low areas often offer the greatest potential for collapse because of the predominance of solution activity. These areas are usually associated with fracture traces or a predominance of existing sinkholes which have resulted from or produced localized zones of high transmissivity in the underlying limestones.

The area surrounding existing sinkholes and subsurface cavities often have a depressed water table due to the free draining sandy soils and the removal of water through conduits in the limestone. This condition depends upon piezometric elevations associated with the aquifer system. Sinkholes may form lakes because of artesian conditions whereas other sinkholes provide continuous or intermittent flow of surface and shallow subsurface waters to the aquifer. Occasionally, the throat of a sinkhole will become plugged with sediments during periods of low rainfall only to unplug when the ponded water elevation rises and piping action is sufficient to remove sediments restricting the downward flow of water.

The cavity size, location with respect to stratigraphy, and many other factors contribute to activity and potential for collapse. Piping and surcharge of the cavity roof induced by surface and subsurface drainage seems to be the prevalent triggering mechanism in agricultural areas. Substantial drawdown of the water table, combined with overhead irrigation for freeze protection of citrus and other crops, often results in an unusually high occurrence of sinkholes.

Although developing sinkholes are considered a catastrophic failure, they usually require hours to several days to fully develop and stabilize. Small diameter sinkholes (< 10 m) may produce a shaft-like opening because of the cohesive character of soils and the potential for arching action. Larger sinkholes are often accompanied by subsequent slope failures which enlarges the sinkhole with greater potential for damage to adjacent structures. In some instances, a collapse results in a minor amount of displacement making it difficult to identify the limits of subsidence. An example of this type of failure was encountered on a segment of the interstate highway.

Subsidence on Interstate 75

On June 8, 1983 a motorist traveling south on Interstate 75, just north of Gainesville, Florida, observed cracks in the pavement and a noticeable depression. Upon being advised of the problem, the Florida State Highway Patrol and the Department of Transportation immediately inspected the site and diverted traffic from the south bound lane. Visual observations indicated that subsidence continued overnight.

Crack location measurements and a visual inspection of the area were performed on June 9th to ascertain the extent and potential cause of subsidence. Figure 2 shows the crack pattern in relation to the pavement and concrete pipe culvert. Cracks in the soil were difficult to locate in the vegetated area west of the pavement and appeared to be discontinuous. Mapping of the observed cracks and the projection of these cracks produced an elliptical failure pattern having a width of 45 meters and a length of 60 meters. The maximum displacement was slightly less than 1.0 meter on the pavement but only 0.1 meters at the crack located outside the fence line (R.O.W.). The culvert was displaced slightly, resulting in a 3.7 centimeter opening at one of the concrete pipe joints.

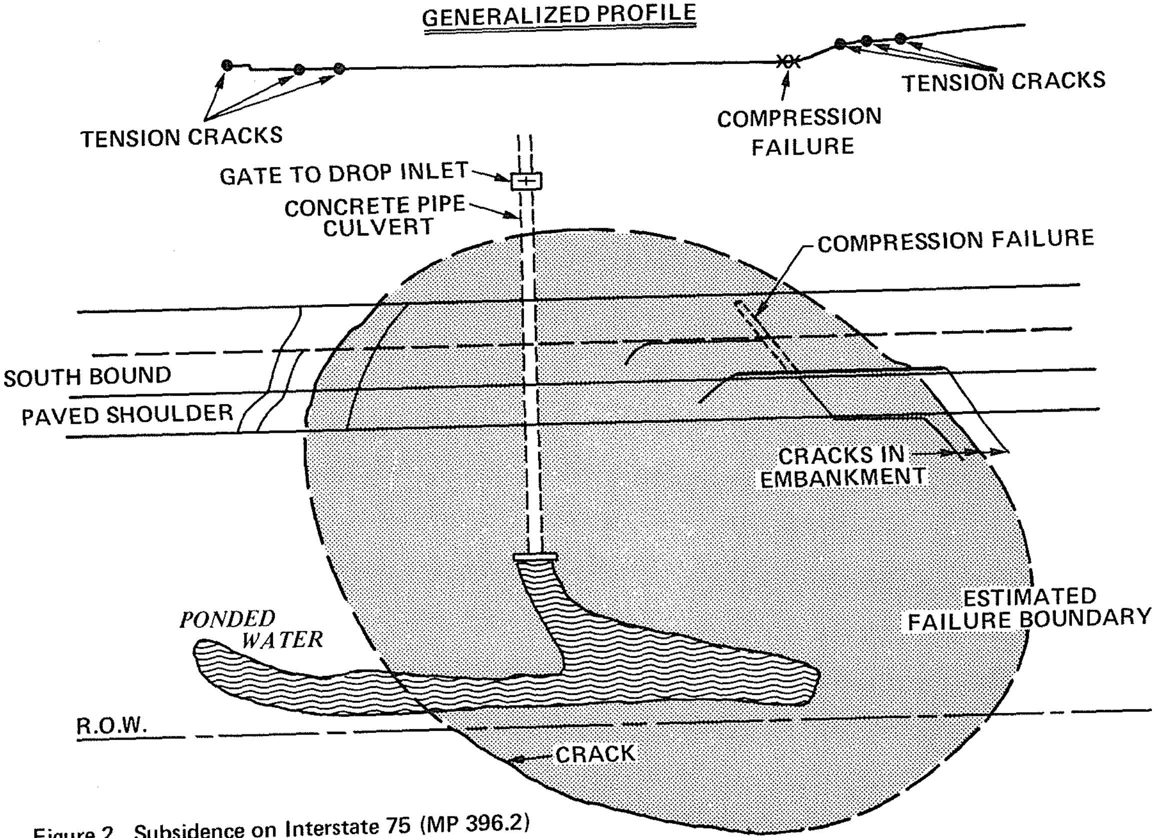


Figure 2. Subsidence on Interstate 75 (MP 396.2)

A drilling rig was used to obtain auger and wash borings to identify soils, stratigraphy, and water table. Over eighty borings were obtained at depths ranging from 12 to 45 meters in an effort to define the exact cause of subsidence. A small cavity was encountered near the drop inlet at a depth of 13 meters which was filled with four cubic meters of grout.

Profiles were prepared from boring data to depict subsurface stratigraphy along the median, pavement, ditch, and west side of the R.O.W. fence. Figure 3 illustrates the highly variable rock contact surface for borings along the median. In some cases, the borings were terminated before rock was encountered. The fracture and/or solution features corresponding to the sharp dips in the limestone are devoid of any tan clayey sand deposits. It is apparent that prior solution activity and piping of soils has resulted in the filling of these depressions with the surficial dark brown sands.

The borings indicated that the depth to the limestone increased substantially toward the west. One boring obtained west of the fence and failure boundary encountered rock at 45 meters. The southwesterly trend of the subsidence zone was aligned with a small, well defined fracture or solution valley which concentrated surface and subsurface water from the upland to the culvert and subsurface, respectively. It is probable that the majority of the subsurface drainage was directed into the failure zone which is in a topographic low. The tension cracks which developed longitudinally in the asphalt pavement indicated that the subsidence was centered to the southwest of the pavement, possibly outside of R.O.W.

A ground penetrating radar survey was performed to define the shallow subsurface along the median. Comparison of radar profile to the stratigraphy illustrated in Figure 3 indicated close correspondence to the depth of the tan clayey sand. The high return from this soil provided good delineation of sand-filled cavities. The maximum depth of penetration obtained with the radar unit was about 11 meters.

An electrical resistivity survey was performed using a polar diapole configuration. Numerous zones of low resistivity were identified such as those shown in Figure 3. The position of low resistivity seems to correlate with the tan clayey sand deposit flanking the cavities. Soil moisture content samples were not obtained so it was impossible to determine if these anomalies were due to higher moisture content of brown sands in the cavities or the clayey sand. Several high resistance zones were located outside and along the failure boundary at an indicated depth of about 12 meters. Borings have not been obtained to verify if these are voids or filled cavities.

Location of High Risk Areas Using Remote Sensing

Satellite imagery and aerial photography have proven useful in geological studies and for engineering uses such as terrain analysis. Although satellite imagery has improved, its resolution is usually inadequate for mapping of small fracture traces or dolines. High quality aerial photography has excellent resolution for mapping of lineaments from aligned depression, sinkholes, etc. This is, of course, a subjective approach to the identification of more active zones for cavity occurrence and collapse.

A site near Dover, Florida was investigated because of the excellent documentaiton obtained on well drawdown and location of recently developed sinkholes. In January 1977, during a period of subfreezing temperatures, farmers began pumping from wells to protect their strawberry crops from frost damage. An observation well indicated a three meter drawdown about

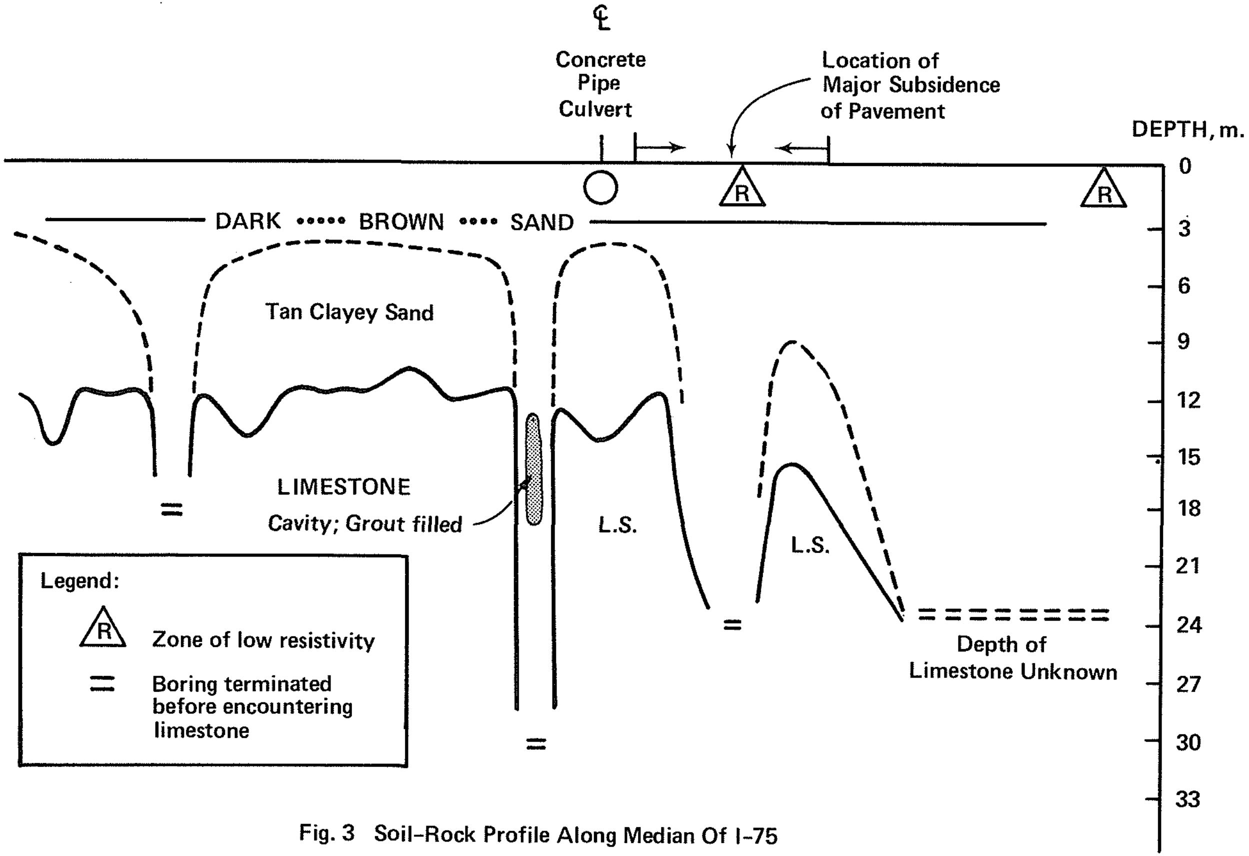


Fig. 3 Soil-Rock Profile Along Median Of I-75

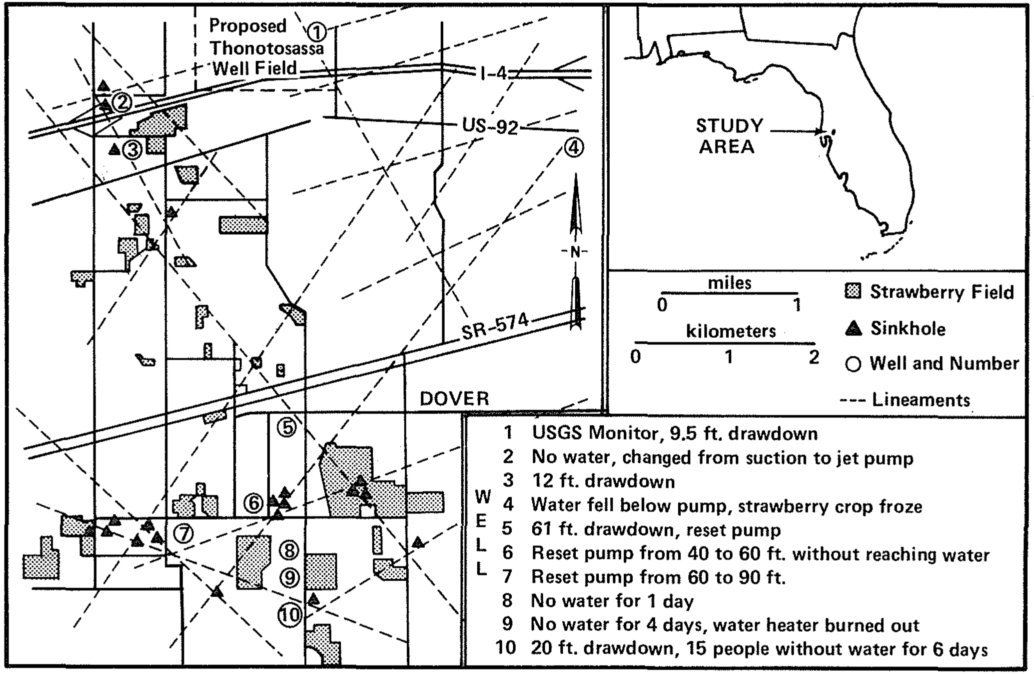
four days after pumping was initiated. It was reported that other wells had attained a drawdown of 18 meters. During this time, 22 sinkholes occurred which were documented by Hall and Metcalfe (1980). This area is described by Brooks (1982) as having sluggish surface drainage and many karst features in a valley from which much of the clastic sediments have been removed. The stratigraphy consists of three to six meters of surficial sands, six to 15 meters of clayey sands, phosphoritic sand and gravels, and clayey limestones and dolomites of the Miocene Hawthorne formation which is underlain by limestone.

Agricultural Soil Conservation Service (ASCS) photos were used to map lineaments on the basis of soil and vegetation tonal alignments and the linears formed by old sinkholes. The mapped lineaments were transferred to a base map which identified the location of the 22 sinkholes. Figure 4 illustrates the relationship between lineaments and the new sinkhole. The lineaments are similar to the northeast and northwest trends mapped by Vernon (1951). The majority of the new sinkholes occur at or near the intersection of lineaments and in slightly lower or depressed areas of the terrain (Casper, Ruth and Degner, 1981). Other investigators have shown that highly fractured zones in carbonate rocks yield greater quantities of water than non-fractured zones (Lattman and Parizek, 1964; Parizek, 1976; U.S. Dept. of Interior, 1978). Similarily, the intersection of these more permeable zones provide the highest availability of water. Consequently, the rate of solution may be significantly greater than in areas of lower transmissivity. This also significantly increases the potential for piping into subsurface conduits. Therefore, these zones can be considered to have the highest potential for subsurface cavities and for development of sinkholes.

Summary

The accelerated consolidation of clays and organic deposits resulting from lowering of the water table does not pose a serious problem except in those cases where buildings and other structures are within the zone of subsidence. Similarly, cavities and the formation of sinkholes are generally not of importance in rural areas but have a significant impact on more developed areas because of the greater potential for building damage or loss of entire structures. Foundation investigations must be conducted in a very comprehensive manner to minimize the chance of not detecting anomalies and subsurface features which influence the location, design, and performance of structures. There are numerous examples of construction problems being encountered because of undetected pockets of poor soils and cavities. The case histories presented previously are good examples of these problems.

Remote sensing methods, particularly air photo interpretation, are valuable for delineating surficial soils, relating to subsurface conditions, and for rational planning of subsurface foundation exploration programs. The use of lineaments or dolines to identify potentially more active zones in Florida's karst terrain appears feasible within the limits of the subjective approach used in defining photo linears. Although the investigation of several sites have shown the intersections of lineaments correlate well with the location of recent sinkholes, there are many contributing factors. Drawdown of the water table, heavy prolonged periods of rainfall, irrigation, thickness and stratigraphy of overburden soils, and topographic position are among the major factors which contribute to the formation and collapse of cavities. These factors also influence the mechanisms which trigger the collapse.



WELL	
1	USGS Monitor, 9.5 ft. drawdown
2	No water, changed from suction to jet pump
3	12 ft. drawdown
4	Water fell below pump, strawberry crop froze
5	61 ft. drawdown, reset pump
6	Reset pump from 40 to 60 ft. without reaching water
7	Reset pump from 60 to 90 ft.
8	No water for 1 day
9	No water for 4 days, water heater burned out
10	20 ft. drawdown, 15 people without water for 6 days

Fig. 4 Lineament and Sinkhole Map for Study Area West of Plant City, Florida (modified from Hall and Metcalfe)

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THE WINTER PARK SINKHOLE AND CENTRAL FLORIDA SINKHOLE TYPE SUBSIDENCE

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Abstract

The karst topography of Central Florida, with marine calcareous sediments overlain by clays and sands, has experienced much sinkhole activity. During the past 20 years, approximately 70 sinkholes have formed in the two major Central Florida counties (Orange and Seminole). Most occurred in April and May, when rainfall is reduced and aquifer withdrawal is high. Sinkholes are typically small, from a few meters in depth to 10 meters in diameter. An occasional large hole forms, which may cause substantial damage, such as the Winter Park sinkhole. It grew to about 106 meters in diameter and 30 meters deep.

Engineering in potential sinkhole areas involves a careful examination of site conditions, geologic and hydrogeologic characteristics, subsurface soil conditions and construction influences. Prediction that a specific sinkhole will occur is not possible, but engineering can offset potential dangers. Evaluation of sinkholes after the fact can lead to a better understanding and effective corrective measures.

Geology of Central Florida

The Florida Platform, about 480 x 800 kilometers in size, consists primarily of sedimentary deposits laid down in a nearly horizontal manner. The oldest sediments are shallow water marine deposits formed on continental shelves; no sediments formed in deep oceanic basins are recognized. Sediments in peninsula Florida consist essentially of fragmental and pasty marine limestones, sandstones, and shales. An uplift along the longitudinal peninsular spine created an elevated topography, with subsurface longitudinal and transverse fractures, thus allowing subsurface drainage. The latter accentuates cavern formation along the fracturing. At the Winter Park sinkhole site below ground surface at about 27 meters above Mean Sea Level (MSL), lie about 18 meters of loose to dense deposits of sands, clayey sands, and slightly cemented sands (Pleistocene) Below these lie about 27 meters of loose to very dense clayey sands, with silt, shell phosphate and dolomite fragments (Hawthorn formation), bedded on limestone of the Ocala Group. This latter layer overlays the Avon Park limestone formation. Solution cavities occur in the Ocala and Avon Park limestone beds. This stratigraphy has been laid down in successive marine exposures. The geology is similar in the Central Florida region to that cited, with variation in depths to limestone and the groundwater piezometric surface, and surface elevation. Figure 1 indicates the Florida Platform with comments; Figure 2 is a profile of the subsurface conditions at the Winter Park site with comments on the hydrology and the sinkhole.

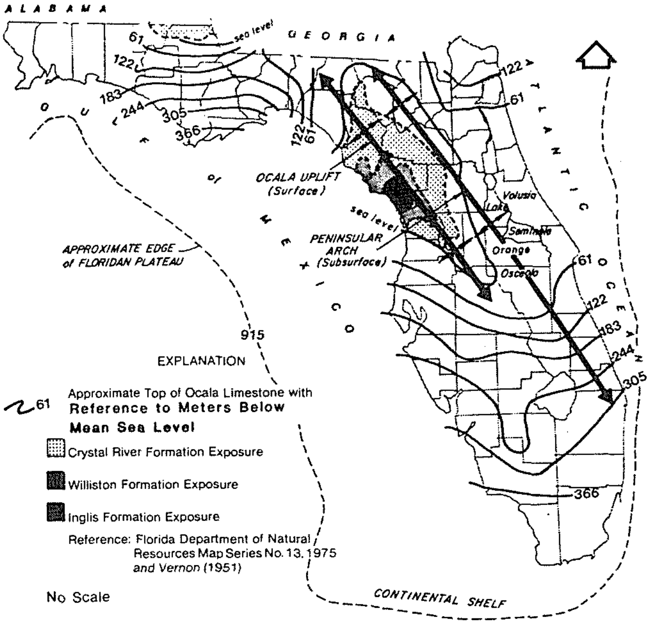


Figure 1. The Florida Platform

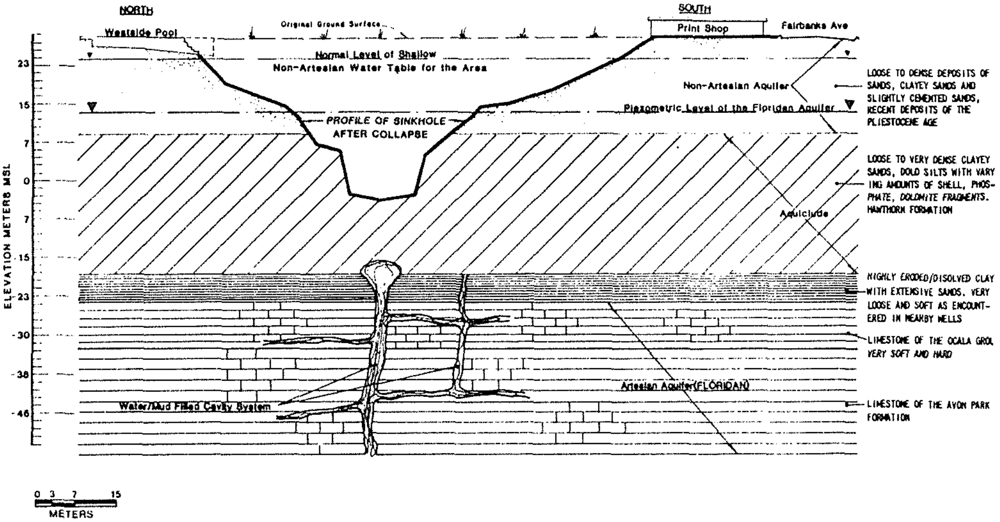


Figure 2. Stratigraphy - Winter Park sinkhole site

Environmental Conditions in Central Florida

Central Florida receives about 130 centimeters of rain each year of which about 70% is lost to evapotranspiration. Thus there is a substantial flow which passes into or over the ground. Since upper levels are sands, extensive infiltration occurs, with recharge of the limestone aquifer through fractures, sinkholes, solution pipes, or porous formations. Western Orange County, in which Winter Park lies, is an excellent recharge area. The Floridan Aquifer in the limestone has a piezometric level of about +14 meters (MSL). It is overlain by the Hawthorn aquiclude. A non-artesian aquifer lies above the Hawthorn.

The climate of Central Florida is subtropical and during geologic ages solution cavities have formed. Subsurface cavern development is extensive as demonstrated by the many large springs and lakes. The countryside is dotted with lakes and cypress bayheads due to various types of subsidence in the past.

Water supply in Central Florida is derived from subsurface wells drawn from the Floridan or overlying aquifer. Thus there are many local disturbances of the subsurface hydrology, which become particularly important in the normally dry months of April and May. Pumpage is a factor of significance in sinkhole formation. In addition, numerous drainage wells, installed over many decades for storm water disposal, affect the local flow patterns as well as the chemical condition of the water inserted into the aquifer.

Local Geology and Sinkhole Activity

In Orange and Seminole Counties subsidence is due to at least five important factors, each vary with the local site conditions.

1. Fault patterns
2. Surface and subsurface drainage patterns
3. Groundwater table fluctuations
4. Soil permeability and its impact on recharge
5. Water chemistry

Winter Park and adjacent areas lie at the terminus of at least three faults. The City is in a topographically closed area which means that stormwater flows into the area. Various water supply wells exist, pumping large volumes, as well as drainage wells delivering corrosive water to the limestone. Recharge does occur, particularly in faulted conditions. Cavities in the underlying limestone, developed over geologic ages, are interconnected with piping into the Hawthorn layer and overlain by sand. Gradual solution and erosion of sands and clays, can lead to surface layer penetration and a sinkhole. Several sinkholes have occurred in the vicinity since 1961 and sinkhole activity is expected to continue. Identified faulting patterns in the area are depicted in Figure 3.

Winter Park Sinkhole

Penetration of the surface sands at the sinkhole site in Winter Park occurred about 8:00 p.m., Eastern Standard Time on May 8, 1981. A resident of a nearby house, which was subsequently lost in the hole, noted a swishing noise and a large sycamore tree disappearing into a hole. Police secured the area overnight. Over the next few days the hole grew to about 106 meters in diameter and 30 meters deep.

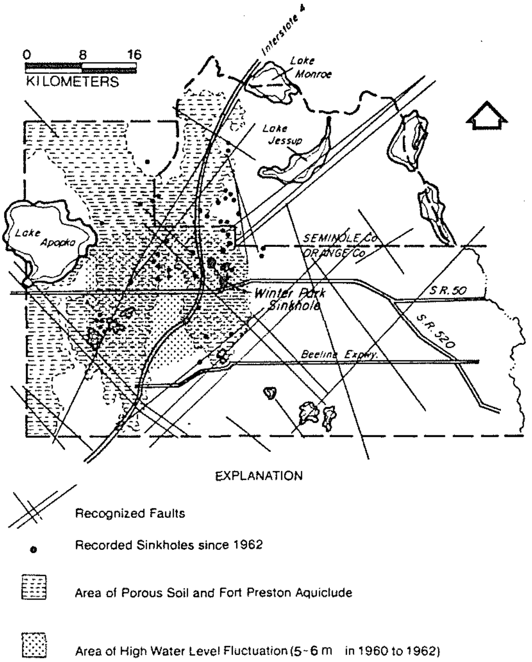
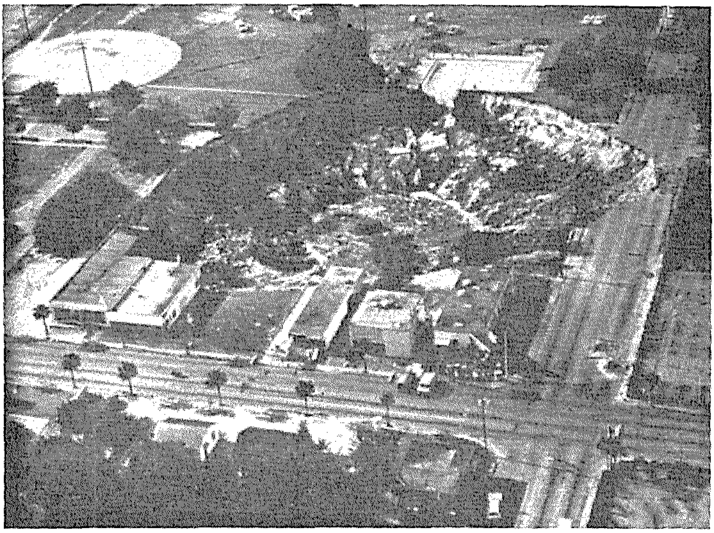


Figure 3. Identified faulting patterns in the area.

Initially it was a dry hole, as can be seen in early photographs. At that point the central aven through the Hawthorn was well defined. Later the hole filled to the piezometric level of the Floridan Aquifer (about +14 meters (MSL)). During formation, the sinkhole destroyed streets, utilities, recreational facilities, one house, several businesses, and six vehicles. It seriously interrupted local traffic. There were no personal injuries. Substantial community impact occurred, with demands on police, city and county governments, and news media. The site required surveillance and restricted access, which continues to this day. Figure 4 is an aerial photograph of the sinkhole at time of maximum growth. Figure 5 shows its location with respect to local sinkhole occurrences in the past.

Investigation - Winter Park Sinkhole

The sinkhole had an immediate detrimental effect on the City of Winter Park, its traffic, businesses, utilities, as well as a resident whose house fell into the hole and was destroyed. As the hole grew it threatened a major traffic artery and interrupted well-traveled City streets. The City was unsure just how much additional damage would occur immediately or over a long period, since the future stability and growth of the hole was uncertain. Hence the City retained Jammal and Associates for initial advice as the hole was growing, then instructed Jammal to investigate and



Orlando Sentinel Star Photo

Figure 4. Aerial photograph of Winter Park sinkhole about 3 p.m., May 11

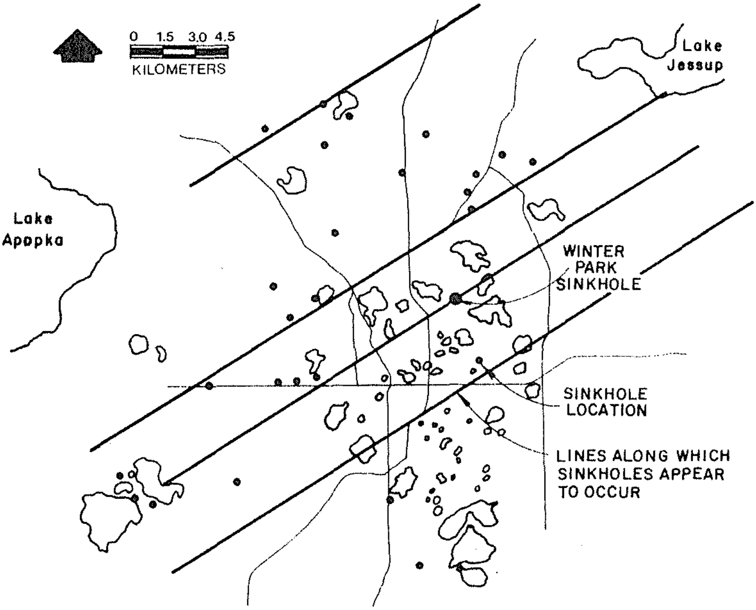


Figure 5. Location and distribution of sinkholes around Winter Park

report on the collapse, the possibility of disastrous long-term effects, and proposals for corrective measures. Jammal investigated the sinkhole site for ten months. A detailed, written report was made to the City.

Investigation proceeded in the following general order:

1. Observations on formation of the sinkhole. A substantial number of photographs were taken (photographs by the news media were available).
2. Monitoring of close-in ground movement and movement of structures.
3. Review of subsurface data for the area.
4. Documenting the topography of the sinkhole by means of photogrametric surveys using color, black and white aerials, and infra-red color.
5. Standard penetration test borings, with four bordering the hole and one inside on a side slope after it stabilized. Initially the slopes were in a "quick" condition.
6. Auger borings to the north and west (toward the nearest lakes, at higher piezometric levels) to identify near surface soil conditions and the shallow aquifer.
7. Soundings and later fathometer readings in the hole to establish the bottom profile.
8. Installing shallow observation wells on north and west lines toward adjacent lakes.
9. Examination of water quality.
10. On-going review of data to adjust the investigative plan as needed.

The sinkhole stabilized with vertical banks in various rim areas and steep slopes in others. These were in surficial sands, (the Ft. Preston layer) hence the sinkhole after stabilizing was still expected to grow substantially due to rain and weathering unless corrective measures were taken. Figure 6 shows the hole and the various boring positions around the initial rim.

The borings indicated that there was a non-artesian surficial aquifer, conically depressed around the sinkhole. Below this, above the Hawthorn, was a secondary piezometric aquifer not interconnected with the surficial aquifer, again circumferentially depressed around the hole. Separated by a few hundred feet, two other areas of depression were found indicating possible future sinkhole sites.

Five standard penetration test borings were made to depths of 52 meters below ground surface. None of the five showed the presence of soft zones that would have indicated potential unfilled voids within the subsoil.

Twenty auger borings drilled to depths of 9 meters provided information on the texture and stratigraphy of the upper sandy soils and allowed determination of the non-artesian water table level. Nothing unusual was encountered for these soils. The primary objective for these borings was to make water table observations and provide guidance for installing the shallow observation wells.

Water was first observed in the sinkhole about noon, May 9, at an estimated depth of 30 meters below ground surface. It slowly rose to the piezometric level of the Florida Aquifer (about +14 meters (MSL)), then continued rising, indicating the aven had been plugged. The water surface was at elevation of +19.5 meters (MSL)

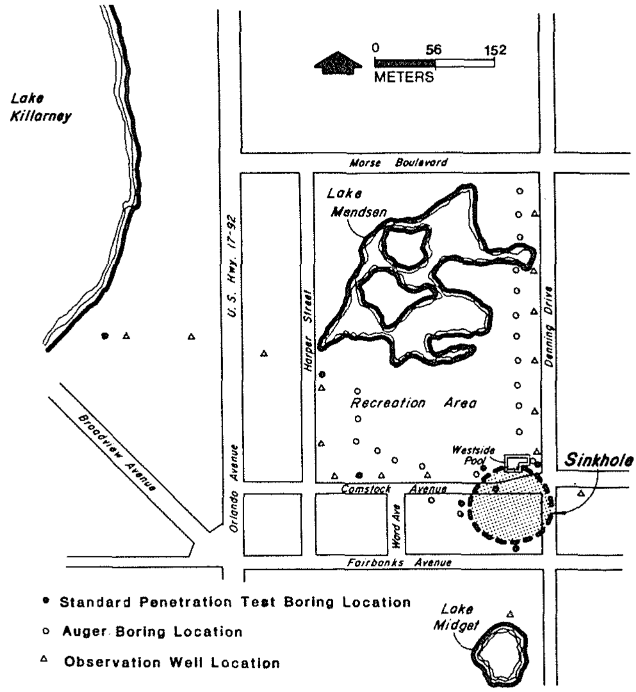


Figure 6. Borings around sinkhole

on July 19, when it suddenly cycled down to piezometric level, +13.4 meters (MSL). Subsequently, it cycled up to +19.5 meters (MSL) and again dropped down to +13.4 meters (MSL) on September 20. Following this second draining the water level rose, indicating a plug, and remained at about a normal lake level in the area. Nearby Lake Killarney has an elevation of +25 meters (MSL). A definite subsurface gradient was observed between Lake Killarney and the sinkhole pool. Fathometer readings and soundings in the sinkhole pool indicated a definite bottom, but the plug was full of debris, e.g., a house, six automobiles, and trees. The sinkhole pool contained extensive floating debris.

Water quality sampling in the sinkhole indicated no unusual results for groundwater.

Technical Conclusions - Winter Park Sinkhole

The Winter Park sinkhole was formed as a result of long-term erosion and ravelling of overburden material into cavernous limestone, not a roof collapse in limestone. Even though the sinkhole developed rapidly, it was a progressive erosion of overburden starting with a small ground depression. The aven was 13 to 17 meters in diameter and by observation penetrated 9 to 12 meters into the Hawthorn formation. It was Jammal's opinion that the sinkhole had been forming over a long time period, but that formation was accentuated over a period of about 50 years, resulting from a progressive decline of the piezometric level of the Floridan aquifer. About 50 years ago this level was about +20 meters (MSL) and now is about +14 meters (MSL). Decline can be attributed both to extended below average rainfall and areal pumping of wells. No evidence was found of contributing factors such as local plumbing leaks. A profile of the sinkhole as it existed and with projected erosion is shown in Figure 7. The sinkhole was stable in 1982 and continued stable through 1983.

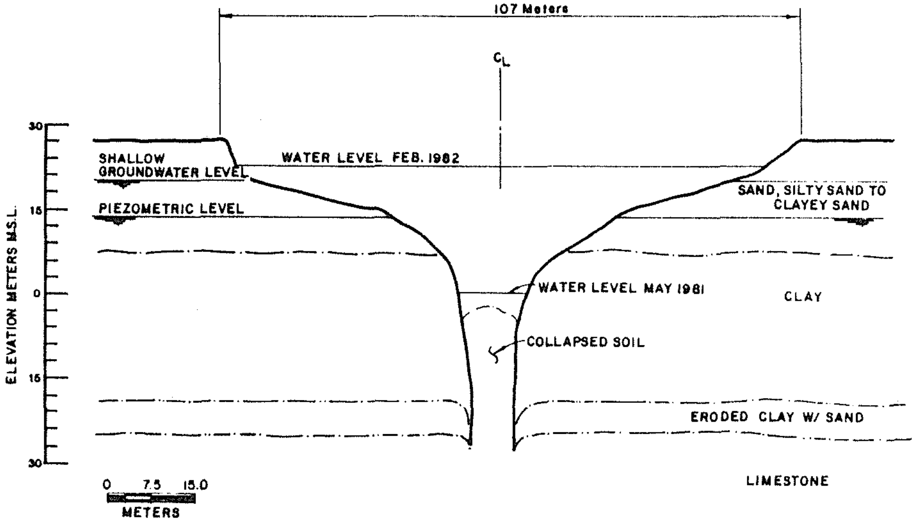


Figure 7. Profile of the Winter Park sinkhole

Alternatives for Correction of Damage

The City was interested in alternatives to offset damage. Correction would be complex since the sinkhole was in part on City owned property, and in part on the private property of several owners. By law, the City could not expend public funds on private property.

A total of seventeen alternatives were considered in the report to the City. Alternatives included bank stabilization to prevent further erosion, a drainage well for lake level control, restoration of Denning Drive, (a major street) and recommendations on a systematic investigative program to document sinkhole changes in the future. The alternatives were expensive, so action was delayed. Through 1983 the private owners commenced bank filling to stabilize and recoup their business property. The City had previously closed several businesses because of dangerous bank conditions. The City initiated construction modifications on their property--steep banks and Denning Drive. Complete filling of the sinkhole was initially rejected because of the amount needed, about 150,000 cubic yards, and cost. To date, substantial filling of the edges has occurred using construction debris. Hence, filling may be the final solution.

Impacts

There was intense media coverage extending internationally. Thousands of visitors observed the sinkhole and crowd control and safety burdened the City. Beyond the public information aspect, there was increased demand for sinkhole speakers both in schools, professional societies, service clubs, and other organizations. Further, it was apparent that, while many agencies were involved, no central depository existed for sinkhole or relevant geologic and hydrologic data. Therefore, a Sinkhole Research Institute was established at the University of Central Florida, Orlando, by the State of Florida. Lastly, it rapidly became apparent that sinkhole insurance coverage was deficient in Florida. Only the family home lost in the Winter Park sinkhole was covered against damage. All other losses were not protected. The 1981 Florida Legislature rapidly redressed this situation, expanding the availability of insurance coverage.

Engineering in Potential Sinkhole Areas

The consulting engineer, particularly in geotechnical engineering, is constantly faced with the potential for sinkholes throughout Central Florida. Since this is a fact of life subsurface investigations and engineering must be approached cautiously. Based on existing information, the future occurrence of an individual sinkhole on a proposed construction site cannot be predicted with certainty. However, the site can be evaluated for potentially dangerous areas, which may be avoided during land planning. Or, if it is necessary to develop a less than perfect site, subsurface investigations must be thorough enough to allow engineering to a reasonable risk.

Investigation of a site requires a systematic examination of the features outlined below.

1. Site Conditions
   - a. Topography.
   - b. Site surface features. Examine for depressions, leaning trees or poles, stressed vegetation.
   - c. Drainage patterns.
   - d. Occurrence of sinkholes, on site or in the region.
2. Geologic and Hydrogeologic Conditions
   - a. Geology of the area.
   - b. Fault lines and lineament maps.
   - c. Groundwater levels.
   - d. Quality of groundwater in the Floridan Aquifer.
3. Subsurface Soil Conditions
   - a. Stratification.
   - b. Consistency - Look particularly for soft zones in drilling and loss of drilling fluid.

c. Texture.

d. Groundwater level - Search for cones of depression.

4. Influence From Construction

a. Wells, drains, and storage ponds may enhance potential in an otherwise stable situation.

Once a thorough evaluation is made, the engineering will be focused on offsetting possible effects, whether by using piling, spread footings, grouting subsurface formations, or similar techniques.

Beyond engineering for initial construction of larger facilities, is the evaluation and correction of damage once subsidence starts, or a hole occurs. This damage may be due to a sinkhole in open land or one impacting a structure. Florida has many small sinkholes. When these stabilize the typical practice is to fill them. Further, there are many structures where the cost of geotechnical investigation and the low frequency of sinkhole formation do not warrant initial detailed analysis such as roads and subdivisions. The latter may be as large as several thousand houses and investigation under each is not practical nor economical. In cases such as these, reconstruction becomes a way of life, or damage can cause abandonment in severe cases. Where cracking starts, indicating subsurface movement grouting has been a common solution. In many cases in subdivisions, the home owner has caused the problem by improperly installing an irrigation well. Post construction tasks are varied and are approached on an individual evaluation basis.

Summary

In Central Florida, geologic history has created a karst environment with great potential for sinkhole development. Sinkholes are a way of life as evidenced by the many lakes, springs, and sinks, and the recent documentation of the phenomenon in more detail. In this region the geotechnical engineer must be particularly competent and vigilant in dealing with potential problems on development sites, and in correction of damage after occurrence.

FRACTURE MAPPING AND GROUND SUBSIDENCE SUSCEPTIBILITY MODELING IN COVERED KARST TERRAIN: THE EXAMPLE OF DOUGHERTY COUNTY, GEORGIA

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Abstract

Subsidence susceptibility maps of a covered karst terrain in southwest Georgia have been developed using a geographic information system DBMANG/CONGRID. Dougherty County was partitioned into 855 cells each 1.18 km^2 in area. Five cell variables were used in the modeling: sinkhole density, sinkhole area, fracture density, fracture length, and fracture intersection density. Broadly similar subsidence susceptibility models were developed from cell data by intersection, and separately by linear combination. In the intersection technique cells having specified values for all variables were located and mapped. In the linear combination technique a map value MV = $W_1 r_1 + . . . W_n r_n$, where W is an assigned variable weight, and r an assigned value weight, was calculated for each cell.

Introduction

The Dougherty Plain of southwest Georgia is underlain by upper Eocene Ocala limestone, which is covered almost everywhere by Oligocene to Recent Surface residuum up to 50 m in thickness. The area is a covered karst region with numerous dolines, uvalas, semi-blind and blind valleys, sinking streams, and springs. Closed depressions have developed by subsidence and/or suffosion of residuum into cavities in the underlying limestone.

As a result of severe droughts during the 1954 and 1977 growing seasons, agriculture in the Dougherty Plain has become increasingly dependent upon ground water from the Ocala aquifer for irrigation. In 1910 less than 8 million m^3 of water were withdrawn for irrigation, in 1977 more than 150 million m^3 were withdrawn. In Alabama an estimated 4,000 man-induced sinkholes or related features have formed since 1900, most of them due to a decline in the water table (Newton, 1977). Increased use of the Ocala aquifer, therefore, even if it does not ultimately lower the regional piezometric surface, could accelerate sinkhole development across the Dougherty Plain near cones of depression produced when irrigation wells are in use.

An attempt has been made to develop maps of ground subsidence susceptibility, which might be of use to land use and water resource planners, by using easily available sinkhole and bedrock fracture data. A sample area - Dougherty County - was selected for study.

The Study Area: Dougherty County

Dougherty County is approximately 43 km E-W, 21 km N-S, and covers 845 km^2. It is underlain by Cretaceous to Recent sedimentary rocks, which dip southeastwards at 2 m/km, these rest on crystalline basement rocks and older Paleozoic sediments. Only the extreme southeastern corner of the county lies outside the Dougherty Plain topographic province. This area is part of the Tifton Upland and is capped by clays of the Miocene Hawthorn Formation. It is separated from the rest of the county by the Pelham

Escarpment. Elevations on the Tifton Upland reach 100 m a.s.l., on the Dougherty Plain they range from 50-75 m.

The topography of the upper surface of the Ocala limestone in Dougherty County is highly irregular because it has been differentially weathered. The limestone may be less than 15 m thick in the west, where it occasionally outcrops, but increases to more than 75 m in the east. The Ocala is covered almost everywhere by surface residuum averaging 13 m in thickness in the northwest and 19 m in the southeast. Residuum thickness is highly variable and may increase by more than 30 m over a distance of less than 3 km (Wilson and Pickering, 1973). The residuum is sandy, silty clay and contains boulders of weathered siliceous limestone up to 2 m in diameter; it is thought to be derived primarily from the weathering of the Ocala limestone.

The Flint River and other surface streams in the county occupy channels that cut into the Ocala aquifer. For most of the year these are effluent streams but at times of peak flow they may become influent. The Ocala aquifer is recharged through sinkholes and blind valleys. Sinkholes vary from 6-8 m deep and from 150-300 m in diameter. Many are alluviated, some contain perched water bodies. Other depressions with open ponors are sites of rapid recharge and therefore sites of potentially rapid ground water pollution.

In recent years man-induced ground subsidences have become more frequent in Dougherty County, particularly in the county seat Albany. In one incident, Hilsman Park, located in a sinkhole, was selected by Albany city officials as an ideal location for a recreational lake. Clay was hauled in to make an impervious floor and logs and tree stumps were placed at the center of the sinkhole in an attempt to plug it. A well was drilled immediately adjacent to the area and water was pumped into the depression for several days forming a small lake. The lake lasted only a short time, the water eventually drained underground when part of the depression floor, including filling material, logs, and tree stumps, subsided into a subsurface cavity (Wait, 1963). A second subsidence occurred near the Banks Halley Art Gallery in Albany. During heavy rains storm runoff is funneled into a sinkhole in the grounds of the gallery and is then pumped out through sewer lines. On June 6, 1973, during an especially heavy rain, one of the pumps failed and 2-3 m of storm water was ponded in the depression. This water triggered a subsidence only 15 m from the art gallery (Wilson and Pickering, 1973).

Modeling Procedure

The relative susceptibility of an area in Dougherty County to ground subsidence is considered to depend on the number of subsurface cavities in the Ocala limestone, and on the likelihood of subsidence or suffosion of residuum into them. Ogden and Reger (1977) concluded from studies in Monroe County, West Virginia, that areas underlain by the most cavernous rock display the most dolines. They found that the percent of the limestone area in dolines, and the doline density, were useful indicators of areas of potential subsidence. Ford (1964) has demonstrated that in the central Mendip Hills of England the formation of one doline (the "mother") tends to promote subsurface conditions that are conducive to the formation of additional dolines (the "daughters") in the same area. Data on sinkhole density and on the percent area in sinkholes were therefore used in modeling as being indicative of both the number of cavities in the limestone and of the likelihood of further subsidence or suffosion of residuum occurring. In addition, as there is preferential development of solution voids along zones of high secondary permeability because these concentrate ground water flow, data on fracture density, fracture intersection density, and the total length of fractures in an area, were also used in modeling the presence of solution cavities in the limestone.

A geographic information system DBMANG/CONGRID was used in sinkhole and bedrock fracture data analysis (Hokans, 1977). The program DBMANG (Data Manager Grid) was used to build and maintain a grid-format data base. The program accommodates up to 30 variables and any number of cells. CONGRID (Conversational Grid) was used to display grid-format data in grey-scale choropleth map form via a line printer. It is presently dimensioned for 29 variables and a maximum of 20,000 cells.

CONGRID has four map output options: (1) simple variable display (a data file map), (2) intersections of variables, (3) unions of variables, and (4) linear combinations of variables. Cells in the grid-format data base with 3-5 sinkholes form a set, cells with 6-8 fractures form another set. The intersection of these two sets (map option 2) includes all cells with 3-5 sinkholes and 6-8 fractures. The union of these two sets (map option 3) includes cells with 3-5 sinkholes or 6-8 fractures. The linear combination option is used when weighting of variables and values is needed in data analysis. For example, if the decline in the ground water table is considered to be twice as important in triggering sinkhole collapse as the depth of surface residuum, these two variables can be weighted 10 and 5 respectively in modeling ground subsidence susceptibility. Map options (1), (2), and (4) were used in this study.

In order to develop sinkhole and bedrock files in DBMANG, Dougherty County was partitioned into 855 cells in 19 rows and 45 columns. Cell size was 1.0 x 1.1 km.

Sinkhole and Fracture Data Collection

Sinkhole Data

The U.S. Geological Survey 7.5 minute topographic quadrangles show approximately 40% of the sinkholes in Dougherty County, the remainder are too shallow to be depicted on these maps which have only a 10 foot (3.05 m) contour interval. Sinkholes were therefore mapped from February 1973, 1:24,000 scale, color infrared images (NASA Project 1473). Color infrared transparencies were viewed stereoscopically at 2x magnification. Sinkholes were identified by the presence of surface water bodies, from vegetation and soil moisture patterns, and from topographic expression. Sinkhole boundaries were drawn at the break of slope with the surrounding flat terrain. Planimetric control was established by also mapping roads and railway lines. Photographic distortion was removed and sinkhole boundaries were transferred to 1:24,000 scale topographic maps using a Bausch and Lomb Zoom Transfer Scope. In total 1,011 sinkholes were mapped, the mean density for the county being 1.1 sinkholes/km^2 (Fig. 1). DBMANG data files were developed for the number of sinkholes in each cell, and for the percent area of each cell occupied by sinkholes. The maximum number of sinkholes in any cell was 15, 32 cells contained more than 10 sinkholes. Five cells had more than 30% of their area covered by sinkholes, 26 cells had more than 20% covered.

The relationship between the number of sinkholes in a cell and the area occupied by them provides an insight into the evolution of sinkhole topo-

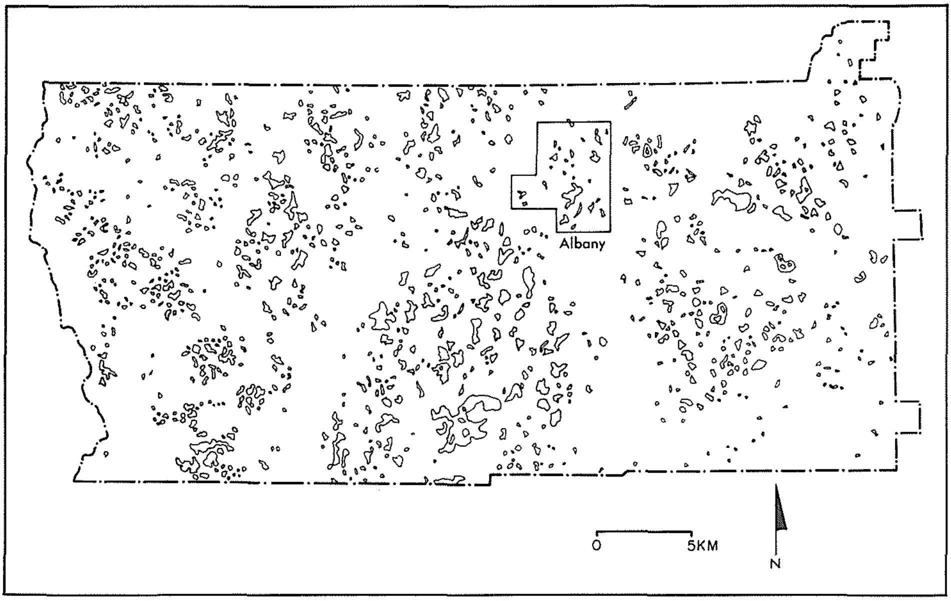

Fig. 1. The sinkholes of Dougherty County, Georgia.

graphy in Dougherty County (Fig. 2.). It suggests that there are two distinct stages in sinkhole development. In the first stage there is a gradual increase in the number of sinkholes in a cell and in the area they occupy. When the number of sinkholes reaches 14-15 a threshold is reached beyond which the topography enters a second stage of evolution. In this stage the lateral growth of sinkholes and their coalescence to form uvalas becomes

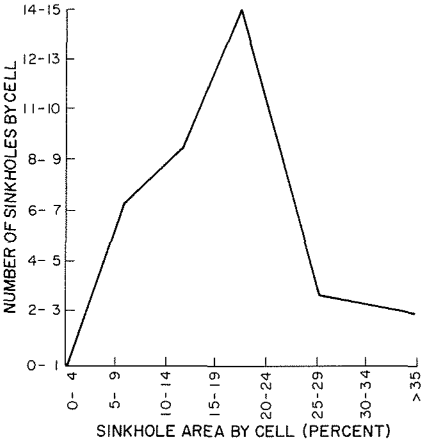

Fig. 2. Relationship between the number of sinkholes in a cell and the area of the cell occupied by sinkholes, Dougherty County, Georgia.

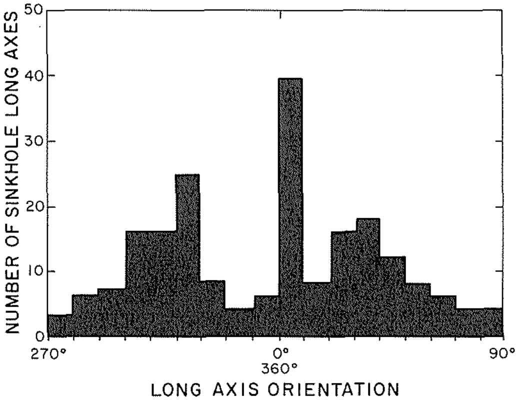

Fig. 3. Number of sinkhole linear shape elements by 10° orientation class for a random sample of 205 sinkholes, Dougherty County, Georgia.

more important than the formation of new sinkholes. The area of the cell occupied by sinkholes continues to increase but the number of separate depressions decreases.

A most important characteristic of the sinkholes in Dougherty County is that many have pronounced linear shape elements. To test whether these show statistically significant preferred orientations the azimuths of prominant long axes or other linear shape elements in 205 sinkholes, in randomly selected cells, were measured and grouped into 10-degree classes (Fig. 3). Chi-square analysis was used to test for non-randomness in the distribution (Pincus, 1953). Six classes, their midpoints at 325, 315, 305, 5, 25, and 35°, were found to be significantly non-random at the 0.05 level. When adjacent significant classes were grouped, three preferred orientations emerged at 315, 5, and 30°.

Wilson (pers. comm.) reports joint directions at 300°, 10°, and 30° in Ocala limestone visible along the banks of the Flint River in Dougherty County during periods of low flow. The close agreement between measured joint directions and the orientations of linear sinkhole shape elements suggests that sinkholes in Dougherty County have developed above, and are elongated parallel to fractures in the underlying Ocala limestone. As the sinkholes have formed by subsidence or suffosion of surface residuum into subsurface cavities, this implies that joints and faults are also the major avenues of ground water movement and solution.

Fracture Data

The distribution and shapes of sinkholes in Dougherty County were used to map possible fractures in the Ocala limestone. Mapping was completed in three stages. In the first stage all pronounced sinkhole long axes and other linear shape elements were identified and marked. In the second stage linear shape elements were connected where these appeared to lie along a single fracture. In addition, fractures were drawn where several sinkholes fell along a straight line. In the final stage of mapping the color infrared images of the county were examined for additional evidence of fractures in the underlying bedrock and for evidence which might suggest modifications to the fracture map prepared from sinkhole data. In total,

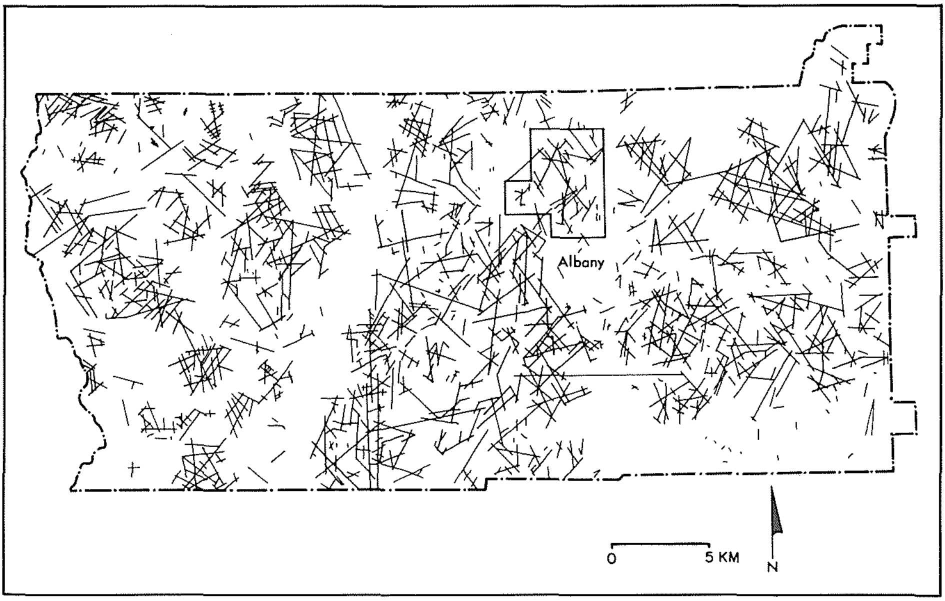

Fig. 4. Possible fractures in the Ocala limestone mapped for sinkhole data, Dougherty County, Georgia.

1,298 possible fractures were mapped the mean length being 1.9 km/km^2 (Fig. 4). DBMANG data files were developed for the number of fractures, the number of fracture intersections, and the total length of fractures in each cell. Thirty cells had more than 9 fractures and 276 cells more than 5 fractures; 155 of the 855 cells had no fractures. Three cells had more than 15 fracture intersections and 20 cells more than 4.5 km of fractures.

In an attempt to explain the fracture pattern in Dougherty County fracture end point coordinates were digitized and lengths and orientations calculated. Fractures were then grouped in 10-degree classes and the number and total length of fractures in each class estimated (Fig. 5). Chi-square analysis was used to test the non-randomness of these distributions (Pincus, 1953). In both data sets the same six classes, their midpoints at 315, 325, 335, 5, 35, and 45°, were significantly different from random at the 0.05 level. When adjacent classes were grouped three major preferred fracture orientations emerged at 325°, 5°, and 40°.

Preferred fracture orientations in Dougherty County agree well with those measured in nearby areas of Florida and Georgia. Vernon (1951) recognizes a fundamental regional pattern of two systems of fractures trending NW-SE and NE-SW in northern Florida and parts of southwest Georgia. Work by Ellwood (pers. comm.) in Climax, Blowing, and Glory Hole Caves beneath the Pelham Escarpment, near Cairo, Georgia, 75 km south of Albany, has revealed two preferred passage orientations at 317° and 48°. Fault planes have been identified in Climax Cave at both orientations suggesting that passages have formed along a conjugate set of shear faults. Ellwood has tentatively interpreted the fracture directions in terms of a stress distribution with the axes of greatest and least stress horizontal and oriented at approximately 90° and 360° respectively, and the axis of intermediate stress vertical.

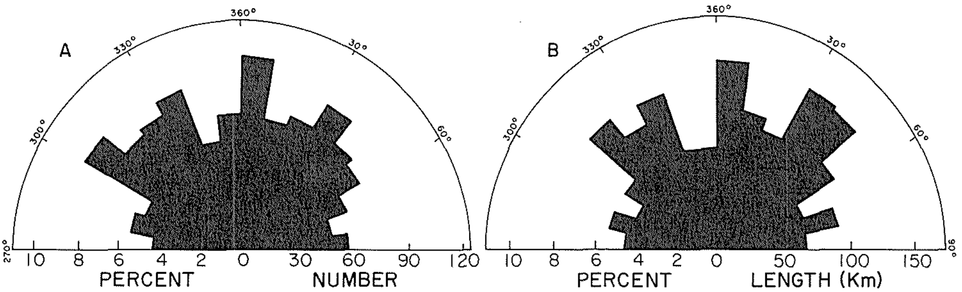

Fig. 5. Number (A) and length (B) of mapped fractures by 10° orientation class, Dougherty County, Georgia.

The fracture sets in Dougherty County were most likely produced by a maximum stress from the west acting against the Chattahoochee Anticline, trending 350° in basement rocks west of Dougherty County. By this interpretation, fractures at 5° are extension fractures, those at 325° and 40° a conjugate set of shear fractures. The broad range of the 325° and 40° fracture directions suggests that they may have been produced by a residual stress field in basement rocks, which caused upward migration of structure features and their impressment on the younger sediments.

Subsidence Susceptibility Models

The sinkhole and fracture data files in DBMANG were used to model via CONGRID the relative susceptibility of cells in Dougherty County to ground subsidence. Separate models were produced by intersections and by linear combination of the five variables. The susceptibility of a cell was assumed to increase with an increase in all variables except sinkhole area. For this variable susceptibility was assumed to reach a maximum when 15-24% of the cell is occupied by sinkholes. This assumption is based on the observation that when 20% of the cell area is occupied by sinkholes further development is dominated by laterial growth and coalescence of sinkholes rather than by the development of new sinkholes.

Intersection

Intersection modeling of susceptibility involved the use of CONGRID to identify and map cells with specific values of the five variables. Four intersections were mapped. A broader range of values for each of the five variables was specified for successive intersections (Table 1). This meant that each intersection identified cells included in the previous intersection plus a number of additional cells. These additional cells were considered to be less susceptible to sinkhole development than the cells already identified. In the first intersection all cells having 12-15 sinkholes, 7-9 fractures, 12-19 fracture intersections, 3.7-7.2 km of fractures, and 15-24% of the area occupied by sinkholes were identified. Only one cell had all of these characteristics and by the criteria established it is the cell most susceptible to future sinkhole development. Intersection 2 identified cells with 10-15 sinkholes, 5-9 fractures, 8-19 fracture intersections, 2.8-7.2 km of fractures, and 10-29% of the area occupied by sinkholes. Fourteen cells were identified, 13 more than were identified by intersection 1. Intersection 3 added 142 cells to those identified by intersection 2, and intersection 4 added 127 cells to those identified by intersection 3; specified

VARIABLE	SPECIFIED VALUES OF VARIABLES			
	INTERSECTION 1	INTERSECTION 2	INTERSECTION 3	INTERSECTION 4
Number of Sinkholes	12-15	10-15	4-15	2-15
Percent Area of Cell Covered by Sinkholes	15-24	10-29	5-35	5-35
Number of Fractures	7-9	5-9	3-9	1-9
Number of Fracture Intersections	12-19	8-19	4-19	2-19
Length of Fractures (km)	3.7-7.2	2.8-7.2	1.9-7.2	1.0-7.2

Table 1 Values of Variables Specified for Intersections 1-4

values for intersections 3 and 4 are given in Table 1. Of the total 855 cells, 577 were not identified by intersection 4. These cells are considered to be the least susceptible to future ground subsidence (Fig. 6).

Linear Combination

In linear combination modeling the variables and the values for each variable were weighted according to their judged influence on the suscepti-

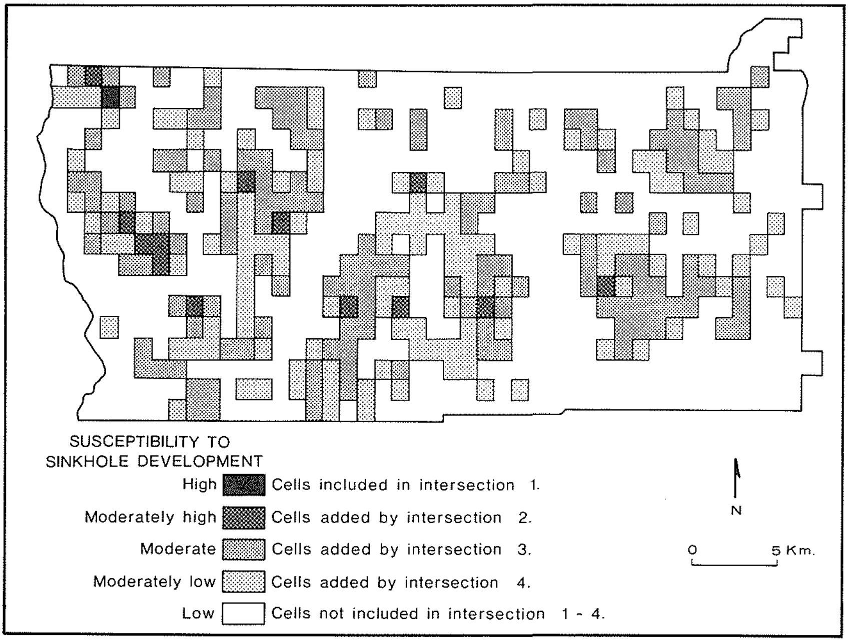

Fig. 6. Intersection model of ground subsidence susceptibility, Dougherty County, Georgia

Variable	Variable Weight	Values (V) and Value Weights (VW)									
Number of Sinkholes	20	V VW	0-1 1	2-3 4	4-5 4	6-7 6	8-9 6	10-11 9	12-13 9	14-15 9		
Percent Area Covered by Sinkholes	6	V VW	0-4 1	5-9 4	10-14 6	15-19 9	20-24 9	25-29 4	30-34 4	>35 1		
Number of Fractures	20	V VW	0 1	1 4	2 4	3 6	4 6	5 6	6 9	7 9	8 9	9 9
Number of Fracture Intersections	15	V VW	0-1 1	2-3 4	4-5 4	6-7 6	8-9 6	10-11 6	12-13 9	14-15 9	16-17 9	18-19 9
Length of Fractures (km)	12	V VW	0-0.9 1	1.0-1.8 4	1.9-2.7 4	2.8-3.6 6	3.7-4.5 6	4.6-5.4 9	5.5-6.3 9	6.4-7.2 9		

Table 2 Variable and Value Weights Used in Linear Combination Modeling

bility of an area to future ground subsidence. Each cell was assigned a map value based on the equation:

$$\text{map value} = W_{k_1} r_{k1} + W_{k_2} r_{k2} + W_{k3} r_{k3} + \ldots W_{kn} r_{kn}$$

where w = variable weight

r = value weight

k_{1-n} = index of the variable.

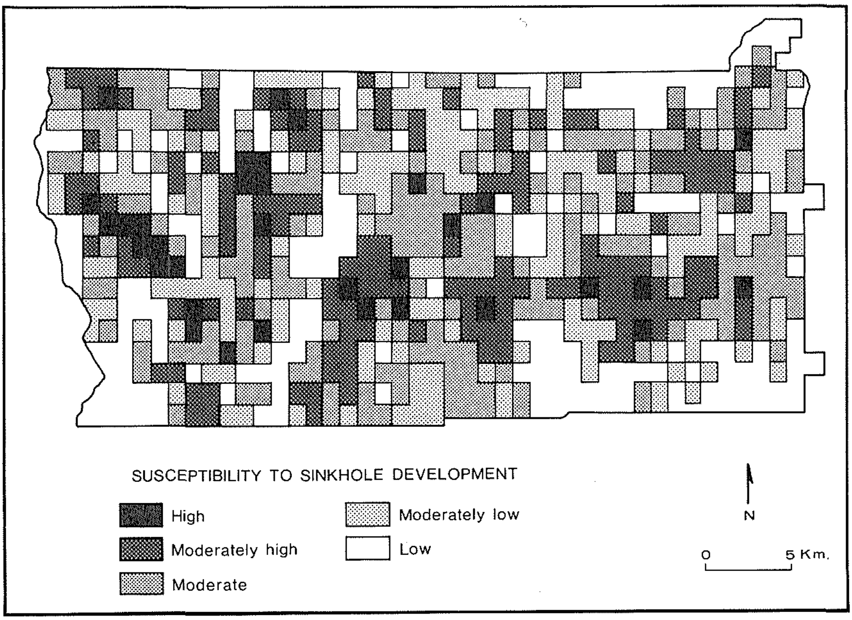

Fig. 7. Linear combination model of ground subsidence susceptibility, Dougherty County, Georgia.

The *linear combination model* was generated using the variable and value weights listed in Table 2. The number of sinkholes and the number of fractures in a cell were considered to be the most important measures of susceptibility to future sinkhole development and were assigned the highest weight 20. The number of fracture intersections was thought to be the next most significant variable, followed by the total length of fractures in a cell; these variables were assigned weights of 15 and 12 respectively. The area of the cell covered by sinkholes was considered the least useful in predicting future sinkhole development and was assigned the lowest weight 6. Value weights for all variables ranged from 1 to 9.

For the variable and value weights shown in Table 2 a cell with values of 5, 3, 7, 6, and 6 for the number of sinkholes, area occupied by sinkholes, number of fractures, number of fracture intersections and total fracture length respectively, would have a map value = (20)(4) + (6)(1) + (20)(6) + (15)(4) + (12)(6) = 338. Map values for each cell were calculated by CONGRID and then were classified into five groups each covering an equal portion of the total range of map values assigned. In a relative sense, these groups of cells were considered to have high, moderately high, moderate, moderately low, and low susceptibility to ground subsidence (Fig. 7). It should be stressed that these terms are relative, cells designated as highly susceptible in the linear combination model may have a different susceptibility to cells designated highly susceptible in the intersection model.

Discussion

Work in Dougherty County has shown that fractures in the Ocala limestone can be mapped from sinkhole data through >50 m of surface residuum. The fracture map of the county (Fig. 4) should be useful in locating high-yield, high specific capacity wells. If all irrigation wells were of high specific capacity this would minimize drawdown and reduce the possibility of ground subsidence. The most likely sites for such wells are at fracture intersections or along single fractures. The fracture map of the county may also prove useful in selection of suitable sites for sanitary landfills. Improperly located landfills have already resulted in contamination of the Ocala aquifer in the Albany area. The most suitable locations for a landfill are those falling in interfracture areas where there is not rapid recharge to the aquifer via underground solution cavities located along fractures. As Figs. 1 and 4 show, however, Dougherty County is a poor waste environment because a relatively permeable residuum with numerous sinkholes overlies a heavily fractured bedrock. The most suitable waste disposal sites are located on the Tifton Upland 15-20 km southeast of Albany, this area is underlain by impermeable clays of the Hawthorn Formation.

The intersection and linear combination models of relative ground subsidence susceptibility are in broad agreement (Figs. 6 and 7). Furthermore, their accuracy is supported by independent data. In both models cells considered most susceptible to ground subsidence correlate with: (1) areas of shallow residuum (particularly less than 10m) where subsidence may be more rapid; (2) two troughs in the piezometric surface of the Ocala aquifer to the west of the Flint River that Wait (1963) feels are areas in which the limestone is cavernous (Fig. 8 A); or (3) regions where the difference between the lowest piezometric surface on record (December, 1977) and the highest piezometric surface on record (March, 1978) exceeds 3 m (Fig. 8 B), in these areas there is a greater loss of hydrostatic support for the residuum during drought periods. These relationships suggest that easily

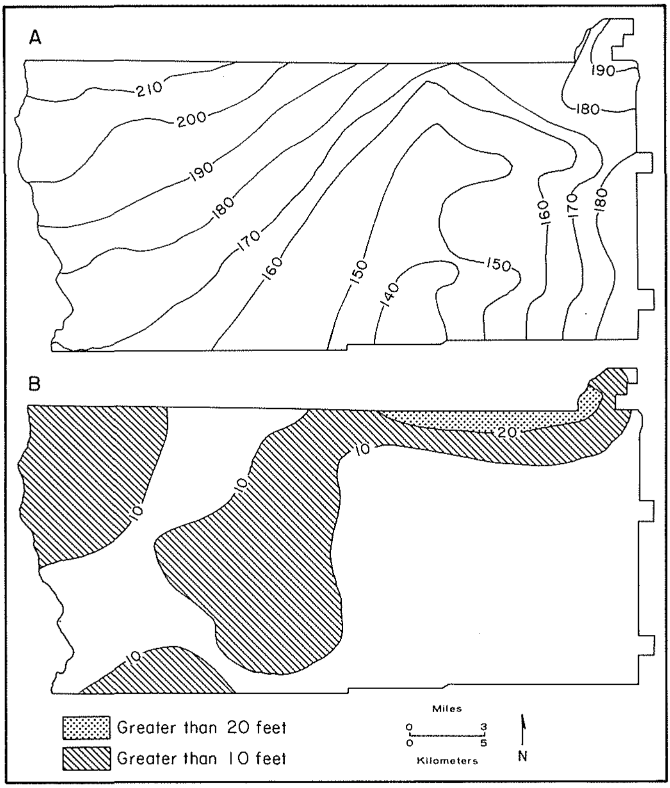

Fig. 8. Form and variability of the piezometric surface in the Ocala aquifer, Dougherty County, Georgia. The piezometric surface of August, 1957 is shown in (A), the difference between the lowest piezometric surface on record (December, 1977) and the highest surface on record (March, 1978) is shown in (B). Elevations are given in feet. Diagram (A) is after Wait (1963), (B) is after Kwader and Wagner (1980).

acquired sinkhole and bedrock fracture data can be used to develop relatively accurate ground subsidence susceptibility maps for covered karst areas.

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SINKHOLE DEVELOPMENT IN SOUTH GEORGIA AND FLORIDA, U.S.A., AND THE FOUNDING OF THE FLORIDA SINKHOLE RESEARCH INSTITUTE

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Abstract

Most sinkholes in Florida and Southwest Georgia are subsidence dolines. They are usually developed in partially consolidated sands and clays overlying the highly permeable Tertiary limestone aquifer. Downward percolation of groundwater through solution channels developed along joints may cause suffosion of the unconsolidated cover. Upward stoping and continued suffosion may eventually cause catastrophic surface collapse. Clay strata perch shallow groundwater and confine groundwater in the limestone, so infiltration is a function of the head difference. Overpumping the artesian aquifer, or the modification of surface drainage, may increase the head differential and trigger sinkhole collapse. Construction activities or heavy rains may also trigger collapse. Man-made ponds and water treatment lagoons, as well as lakes, may drain suddenly through newly opened sinkholes. Because new sinkholes are so common in Florida, insurance protection is offered. The Florida Sinkhole Research Institute was founded in 1983 to coordinate and conduct research on sinkhole development so as to minimize the impact on man.

Introduction and Geologic Setting

The rapid, occasionally catastrophic development of subsidence dolines (locally termed sinkholes or limesinks) in Florida is world famous. It is important to understand, however, that not all sinkholes in Florida develop by the same mechanism. Furthermore, this same natural hazard is also an environmental problem in areas of Georgia and South Carolina where the geologic framework is similar.

South Carolina, Georgia, and Florida are all part of the Southeastern Coastal Plain. Shallow marine sediments have been deposited in an off-lapping sequence on the coasts of both the Atlantic Ocean and the Gulf of Mexico. At the juncture of these two provinces, the Florida Platform extends southerly from the North American mainland. This platform has created a stable, tropical shallow marine environment from the Cretaceous Period to the present.

The Floridan Peninsula is underlain by a continuous sequence of limestones deposited from the Paleocene to the Miocene Epochs (Figure 1). Correlative limestones extend into the Florida panhandle*, Georgia, and South Carolina. However, away from the peninsula the limestones are generally the more marine facies and may grade updip (inland) into clastic continental/littoral equivalents.

The clastic, Miocene Hawthorne Formation caps this sequence in the northern peninsula and coastal plains. However, in many areas of Florida the majority of the Hawthorne Formation is also calcareous or dolomitic. This thick sequence of carbonate rocks constitutes one of the world's most prolific aquifers. In Florida it is called the Floridan Aquifer and elsewhere it is termed the Principal Artesian Aquifer.

Although structural deformation has been slight in the coastal plain, peninsular Florida has been affected by two episodes of anticlinal uplift, both trending

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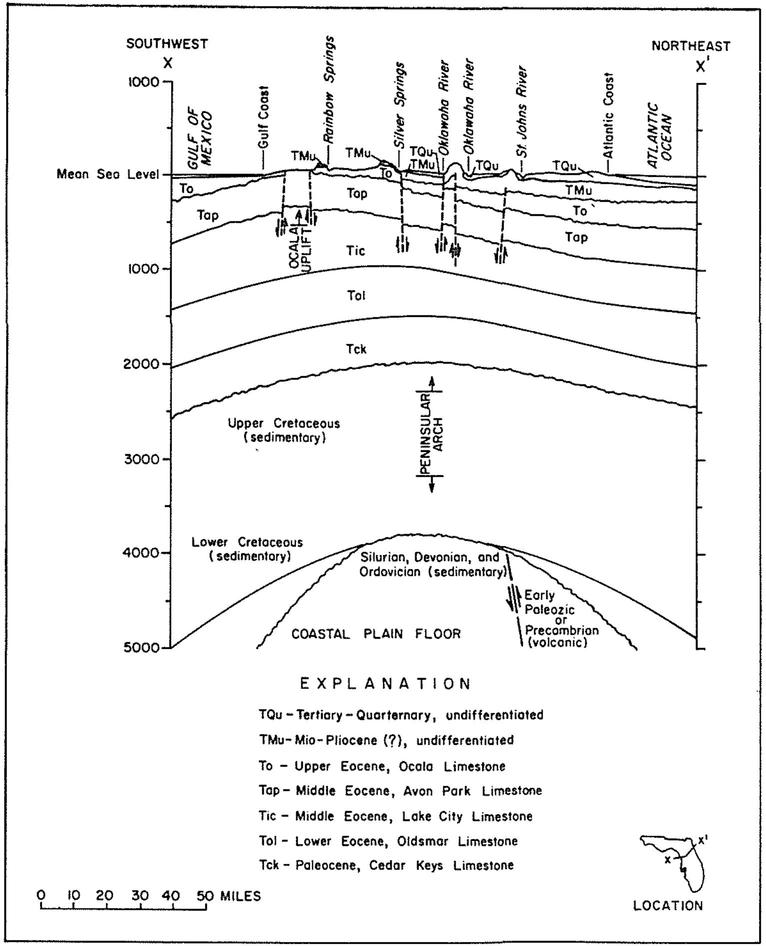
*That portion of the state of Florida extending westward along the Gulf coast.



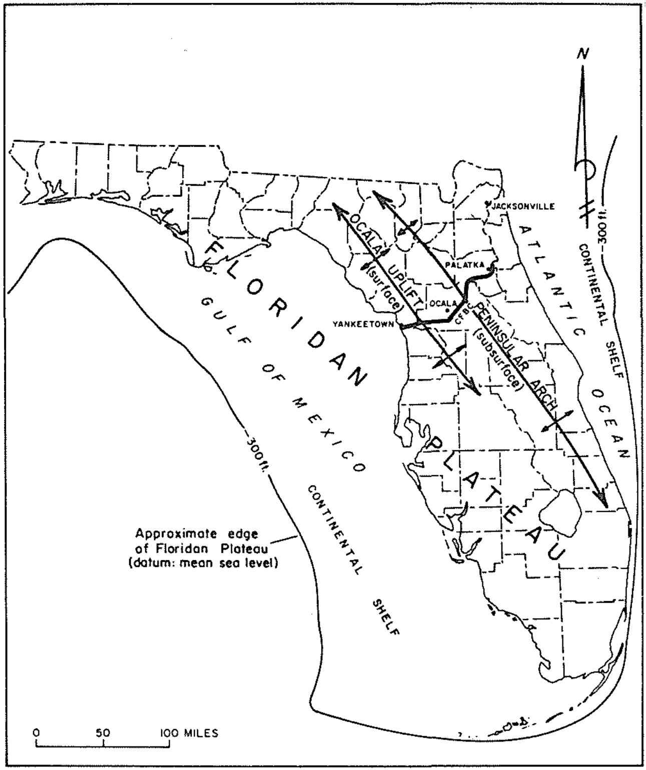


Fig. 2 (above) The Floridan Plateau showing the Ocala Uplift and the Peninsular Arch (from Faulkner, 1973).

Fig. 1 (left) Geologic section of north Peninsular Florida (from Faulkner, 1973).

parallel to the peninsula (Figure 2). The Peninsular Arch is a late Paleozoic structure related to plate movements and the Allegheny Orogeny. The Ocala Uplift formed during mid-Tertiary time. Several related normal faults also break the strata into a series of horsts and grabens (see Figure 1). In addition, prominent fracture sets are detectable as photolinears. These trend generally NW-SE and NE-SW and are probably part of a global trend due to flexing of the crust by tidal and rotational forces (Blanchet, 1957).

In Northern and Central Florida surficial erosion has removed the Hawthorne clastic cover along the western flank of the Ocala Uplift. Plio-Pleistocene coastal terrace deposits, dominantly sandy, thinly cover the exposed limestone. At the contact of the limestone and the more durable Hawthorne Formation, an escarpment is present. This is known as the Cody Scarp in northern Florida and the Pelham Escarpment in Southwest Georgia. It is this feature which divides the area into two distinctly different types of karst areas: a bare (thinly mantled) coastal area and a covered upland area. Near the edge of the escarpment the cover may be breached by sinkholes; in this zone several rivers also sink (Figure 3). In fact, sinkhole subsidence may breach the cover through up to 90 meters of overburden (Jammal and Assoc., 1982).

Along the eastern coast of the peninsula the Hawthorne was not removed prior to deposition of the more recent terraces, so the limestone is thickly covered and karst features are few. Both uplifts plunge to the south and the limestone becomes covered by up to 370 meters of younger sediment. Karst features are not commonly found in southern Florida.

The General Karst Hydrogeology of Florida

The young (Tertiary) limestones of the Floridan aquifer are highly porous. Microporosity arises from primary pore spaces and those related to diagenesis (Randazzo, 1976). Dolomitization due to a migrating seawater/freshwater interface in the groundwater may also produce zones of additional porosity (ibid.). Finally, paleokarst processes which occurred during the formation of unconformities may produce extensive zones of macroporosity, notably at the Ocala-Suwannee and Suwannee-Hawthorne contacts (Burnson, 1981). All of these sources of porosity are present in these Tertiary limestones thus creating a highly permeable aquifer with transmissivity estimated at 45,000 m^2/d in the northern peninsula (Hunn and Slack, 1983), and 11,000 m^2/d updip in the Dougherty Plain in Georgia (Kwader and Wagner, 1980).

Despite the unusually high overall permeability of these limestones, groundwater flow is further facilitated by fracture zones. Thus, the resultant karst features still are joint-controlled. Beck and Arden (1983) have noted joint control in caves in the Tertiary limestones in Southwest Georgia. Ruth and Degner (this volume) have demonstrated that sinkhole locations near Tampa, Florida, are related to fracture intersections. In the same area, Moore and Stewart (1983) concluded that photolinears are zones of increased limestone solution.

Karst dissolution results from the attack of acidic groundwater infiltrating into the limestone. Rainwater absorbs some CO_2 from the air but it absorbs much more from decaying vegetation in the soil. The CO_2 reacts with the water to form carbonic acid. As the acidic water seeps downward, facilitated by fractures in the limestone, it enlarges these fractures into downward tapering vertical conduits, ranging from centimeters to meters in diameter. These are known generally as subsoil karren. Such vertical channels frequently, but not exclusively, form at fracture intersections. In some instances these conduits maintain a nearly constant diameter and a nearly circular shape and are known as solution pipes. In quarry walls it can often be seen that these vertical features extend downward to a horizontal zone of macroporosity where inflowing water can move laterally. These vertical channels are generally filled with the overlying younger sediment which has settled downward, or may have been deposited there. The surface expression

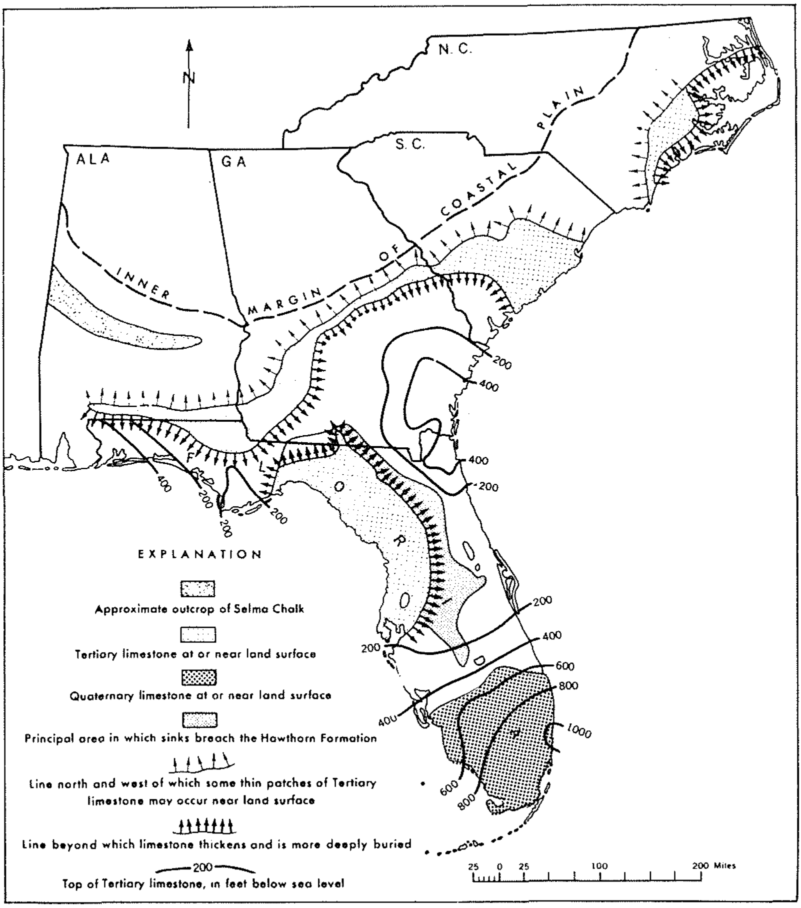


Fig. 3 Map Showing Outcrop Areas of Quaternary, Tertiary, and Cretaceous Limestone. Contours show top of Tertiary Limestone in the Subsurface. (From Stringfield and LeGrand, 1966)

of these conduits are the sinkholes which we will describe in more detail below.

Below the water table the groundwater moves laterally. Horizontal conduits may be localized at specific stratigraphic levels, particularly the zones of macroporosity mentioned previously. Thus, the Peacock Springs Cave System, with nearly five kilometers of underwater passage mapped by divers (Figure 4), is localized at the Ocala-Suwannee contact (Fisk and Exley, 1977).

Where the land surface is incised below the water table, this laterally moving groundwater re-emerges as springs. It is particularly in this area of converging groundwater flow that the enlargement of conduits to cavernous dimensions occurs. The Peacock Springs Cave System has formed at exactly this position. Many other of Florida's springs also debouch from sizeable conduits which have been explored by divers. Radium Springs, in Southwest Georgia, flows from a similar submerged cave (Beck and Arden, 1983).

Thus the general karst hydrogeologic cycle in Florida may be summarized as: (1) infiltration through vertical conduits such as solution pipes and sinkholes; (2) lateral flow through the permeable limestone, particularly along zones of megaporosity or through conduits; and (3) re-emergence at the surface, generally through conduit systems, as spring flow. Note that this may be different from the classical concept of karst in dense, impermeable limestones, where an integrated conduit system must develop from input to outflow.

Sinkhole Formation Today

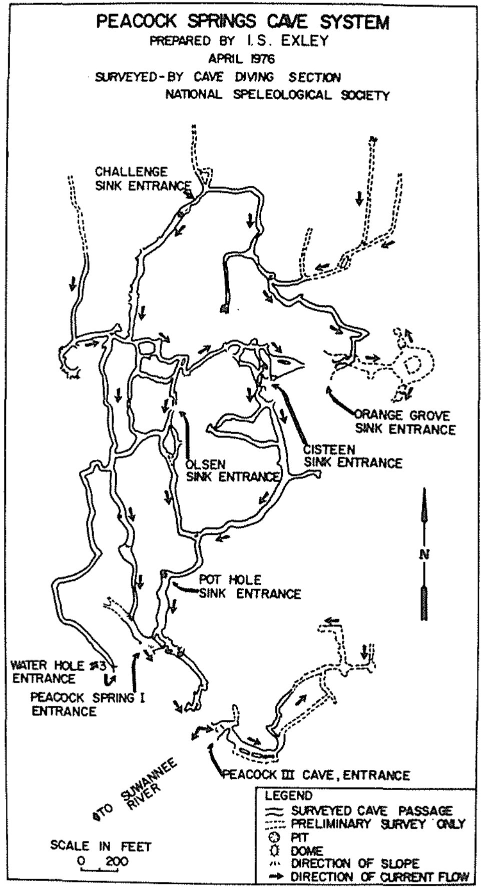
Sinkholes are more properly called dolines (Jennings, 1971). However, in Florida the term "sinkhole" or simply "sink" has become thoroughly entrenched in both the common and professional English language, and it will be retained herein. Sinkholes are "enclosed hollows of moderate dimensions" (Williams, 1964), originating due to the solution of the underlying bedrock. However, the mechanism by which the solution of the bedrock, usually limestone, produces enclosed hollows varies as does the form of the depression and the circumstances of development.

While sinkholes in Florida are, of course, forming today as they have in the past, it is misleading to think of the catastrophic collapses which make newspaper headlines as originating recently. They have not. The sudden subsidence which disrupts the present land surface is only a late stage expression of processes which have been occurring beneath the surface over thousands of years.

It is also important to emphasize that the location of catastrophic collapse sinkholes in Florida (and elsewhere) is not generally due to increased limestone solution occurring presently (i.e., arising from acid rain or groundwater pollution by industrial wastes). The dissolution of the limestone is a slow process. For Florida, Sellards (1908) estimated thirty centimeters of surface lowering in 5,000 - 6,000 years. In an area of the Floridan Aquifer subjected to increased flow due to pumping, Sinclair (1982) estimated thirty centimeters of surface lowering per 1,700 years. Thus, the dissolution of the limestone has taken place over a period of tens, or possibly, hundreds of thousands of years. Further, some of the conduits which are being reactivated today originally formed as part of a paleokarst during Pleistocene, or even Tertiary time. While continued slow dissolving of the limestone may produce broad, shallow solution sinks, which are continuously forming today, their rate of growth is slow in human terms, and the catastrophic subsidence sinkholes owe their sudden birth to other factors.

Several different mechanisms may form sinkholes in Florida. In the thinly-mantled karst terrain, sinkholes may form when the roof of a subterranean cavity collapses. Such collapse produces steep, limestone-walled sinks, frequently containing water. The cenotes of Yucatan, Mexico, are classic examples of this (Jennings, 1971), and are analagous to such sinkholes in Florida. The numerous entrances to the Peacock Springs Cave System in Florida are collapse sinks (Figures 4 and 5). Such collapse sinks are common in areas where the limestone is near the surface. The fact that numerous collapse sinkholes occur today is misleading. Because they are formed in bedrock they maintain their form for a relatively long time. In actuality, most authors agree that roof collapse sinkholes are rare (Sowers, 1975; Newton, 1976; Williams and Vineyard, 1976). A preliminary assessment of the records of recent sinkhole collapses in Florida by the author confirms that collapse sinkholes (in the technical sense) are rare (Figure 5).

Very slow, gradual subsidence of the land surface may occur as the limestone dissolves if the overburden is thin and non-cohesive (sandy). This is the case in much of the thinly-mantled karst area. Solution of the upper portion of the limestone at the intersections of fracture zones will gradually englarge these fractures, widest near the surface where the attack of the acidic water is freshest. However, because the thin overburden totally lacks ohesion, the sand will continually settle into these slowly enlarging joints producing equally gradual lowering of the land surface. The shape of such shallow depressed areas will



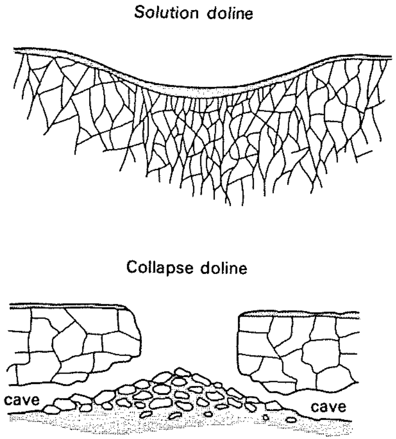


Fig. 5 (above) Two Types of Dolines Found in the Thinly Mantled Karst Terrane in Florida.

Figure 4 (left) Map of Peacock Springs Cave System. (from Fisk and Exley, 1977) depend on the pattern of the controlling fractures. Although such solution sinkholes are not as spectacular as the more rapid collapse, or cover subsidence sinkholes, it is important to note that sinkholes can, and do, form slowly and gradually in some areas of Florida (Figure 5).

In the covered karst terrane, subsidence processes produce much larger, spectacular sinkholes such as the Winter Park Sinkhole (See Jammal and Assoc., 1982; and Jammal, this volume). These are properly termed subsidence dolines according to Jennings (1971) who gives the following description, "Where superficial deposits or thick residual soils overlie karst rocks, dolines can develop through spasmodic subsidence and more continuous piping of these materials into widened joints and solution pipes in the bedrock beneath. They vary very much in size and shape. A quick movement of subsidence may temprarily produce a cylindrical hole which rapidly weathers into a gentler, conical or bowl-shaped depression." Although Sweeting (1973) would term these alluvial dolines or possibly solution subsidence dolines, none of her descriptions so aptly match the Floridan examples as does Jennings' quote above. Sowers (1975) has termed these ravelling sinks and this terminology is prominent among Florida's professional engineers. However, Jennings (1971) term not only predates Sowers (1975), but the study of karst has historically been the realm of geologists and geographers. Therefore, the term "subsidence doline" or subsidence sinkhole seems to have priority.

A simplified example of the subsidence mechanism was clearly revealed in a recent (1983) sinkhole north of Ocala, Florida. At the site there is only approximately three meters of sandy sediment overlying the limestone. This sinkhole which formed rapidly consists of a 3 - 4 meter wide, 3 meter deep, funnel-shaped depression in the sand leading to a 5 - 6 meter deep, 1 - 1.5 meter diameter, almost perfectly cylindrical shaft in solid limestone. A vertical fracture is visible in one side of the limestone pipe. The bottom of the pipe is floored with loose sand and vegetation debris. Numerous similar solution pipes filled with sediment are visible in the walls of the many local quarries. The obvious hypothesis is that this sinkhole formed when the sediment formerly filling the pipe settled, or was washed downward. Surface sediment immediately fell into the open pipe and the sides of the surface opening quickly eroded to a semi-stable angle of repose. Examples such as this are common in the thinly mantled karst area.

The major subsidence sinkholes, however, have a more complicated history. In the covered karst area the clayey Hawthorne Formation overlying the limestone provides a durable, cohesive bridge or roof over vertical channels in the limestone. When the fill in these channels subsides, the clay bridges the void. Over a long period of time the clayey sediment may gradually crumble into the channel only to wash into deeper conduits in the limstone. As this cavity in the clay grows upward, its sides also crumble and it grows wider. Thus, over an extended period of time, a sizeable void may develop in the clay, much larger than the diameter of the underlying limestone conduits. When the void enlarges so much that it nears the top of the Hawthorne Formation sediment, the thin clay roof remaining may suddenly collapse, allowing the overlying sandy sediments to rapidly subside into the large void in the clay. If the limestone conduit is open, unconsolidated surface sediment may continue to erode into the opening and disappear to greater depths (sometimes including cars and houses: Jammal and Assoc., 1982) as the sinkhole widens to stable side slopes. If there is a perched water table in the sand, then the sand/water slurry may flow in towards and down into the conduit and cause rapid widening of the sinkhole (see Figure 6).

In all these processes, downward sediment movement is aided by infiltrating groundwater. However, the clayey, low permeability Hawthorne Formation effectively caps the limestone and perches water in the sands above. Water recharges the limestone slowly through the thick clay, or more rapidly through cracks or sand-filled pipes breaching the clay. Any mechanism which increases the

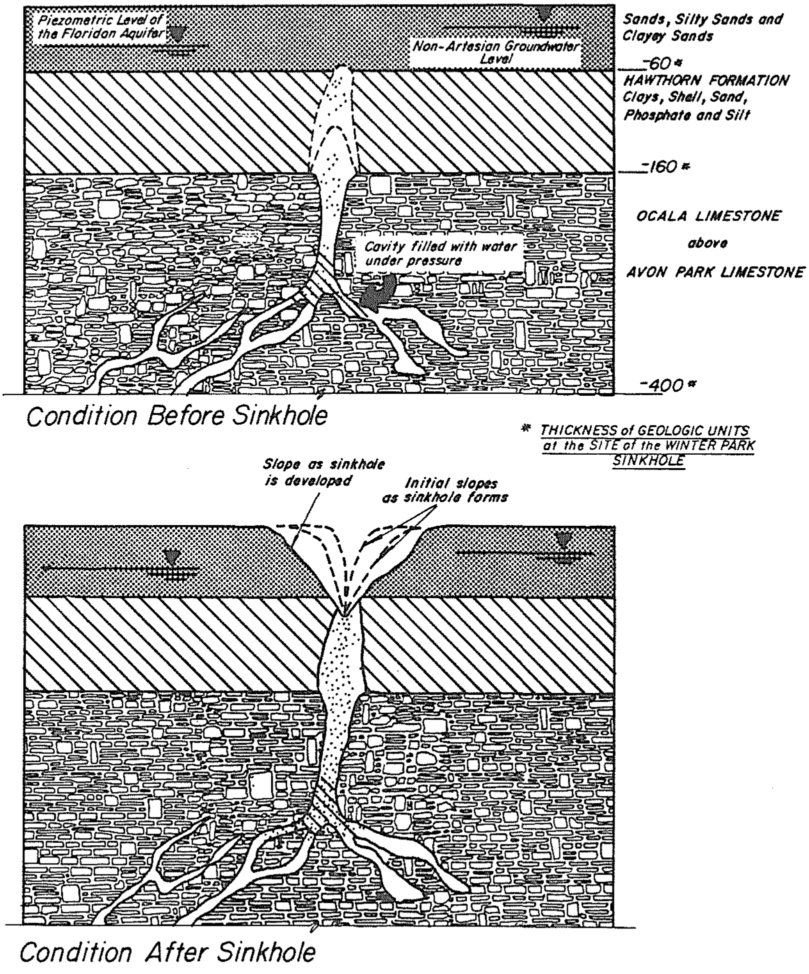


Fig. 6 Development of Subsidence Sinkhole in Covered Karst Terrane in Florida (From Jammal and Associates, 1982).

head difference between the artesian water in the limestone and the perched water in the surface sands will increase the downward flow and the downwashing of sediment and may trigger sinkhole development. The head difference may be increased in two ways: the piezometric surface may be lowered, or the water table may be raised. The relationship of water levels and hydrostatic forces to collapse is complicated by the buoyant support which water may provide to the overlying sediment and the shrinking and cracking which may occur as sediments dessicate.

In many areas of Florida, particularly west-central Florida, most sinkholes form during the dry periods of May and early June (Stewart, 1976). Such a short dry period should lower the shallow water level faster than it decreases the potentiometric surface and thus decrease the head difference. However, increased consumption of water from the limestone may increase the differential, or possibly the artesian aquifer response to drought is more rapid than expected. More detailed study is necessary to fully understand this trigger for sinkhole collapse.

However, in other instances the mechanism is more obvious. In Ocala, Florida, in April 1982, 30 centimeters of rain fell in an afternoon. This obviously caused heavier than normal infiltration into the thinly covered limestone, accompanied by downwashing of sediment causing the formation of numerous damaging sinkholes, at least twelve documented in the newspaper.

During the intense freeze of late December, 1983, agricultural interests used irrigation constantly to protect delicate crops from freezing. Piezometric levels dropped rapidly leaving some of the shallower domestic wells without water for days. During this period a new sinkhole twenty-five meters wide, eleven meters deep, and steep-walled, developed in Pierson, Florida, where cover thickness is approximately thirty meters. An adjacent pond, approximately 90 meters in diameter and three meters deep, drained into the new sink which accepted the water without filling. Witnesses reported seepage emerging on the sides of the sink approximately three meters below land surface. This indicates that the shallow water table was depressed in the sinkhole area during or prior to its formation, probably by initial recharge downward through the developing openings. Local depression of the shallow water table has been noted in association with other sinkhole collapses (Jammal and Assoc., 1982). Over the next week, the sinkhole gradually filled with water to this level, but nearby lake levels indicated that the local water table around the sinkhole was still somewhat depressed. Other examples of sinkhole formation due to irrigation for freeze protection have previously been documented in various areas of Florida (Hall and Metcalfe, 1977; Rugledge, 1980).

Sinkhole Impact

In recent years, the expanding population and man's increasing modification of the natural environment have caused sinkhole collapse to become a major economic hazard in Florida, and to a lesser extent in Georgia and South Carolina. Insurance companies in Florida have begun offering general coverage to all home-owners and specific additional coverage for commercial establishments. However, these claims create a need for scientific input to establish actuarial tables of sinkhole occurrence and even to differentiate sinkhole development from more gradual subsidence (see Ruth and Degner, this volume). Financial contributions from the insurance industry initiated the Florida Sinkhole Research Institute which is now part of the Department of Civil Engineering and Environmental Sciences at the University of Central Florida, in Orlando, Florida. However, the institute's mission is broad-based. In addition to providing input to the insurance industry, the FSRI will develop publications explaining sinkholes to the non-technically educated citizen, will centralize and computerize statewide data on areas and conditions of sinkhole collapse, will sponsor research on methods of detection and repair, and will conduct such other research as may be appropriate. A major symposium will be held October 15 - 17, 1984, in Orlando, to assemble the state-of-the-art knowledge on sinkhole geology and engineering. Although the Florida Sinkhole Research Institute was chartered to serve the citizens f Florida, it is anticipated that in the future, the research centered here will have a national and international impact.

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PHOTOGEOLOGY OF SOME SALT KARST SUBSIDENCES, CHESHIRE, ENGLAND.

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Abstract

Natural and man-made salt Karst subsidences are widespread over the rocksalt districts of Cheshire, England. These subsidences severly affect engineering structures, buildings, roads, railways, canals, rivers, and agriculture.

Photography can record such events, significantly supplementing topographic surveys, thereby substantially assisting in the understanding of the nature and origin of subsidences of the region.

Photogeology techniques applied to the problem are described and evaluated.

Introduction

The nature and problems of the rocksalt districts of Cheshire were described by Howell and Jenkins (1976). In essence the districts contain two main beds of rocksalt of Triassic age, attaining a maximum thickness of some 400m and 190m respectively. These beds occur in a faulted, elongated structural basin some 80km by 50km in extent. The rocksalt does not reach the surface but sub-crops beneath brecciated collapsed marls of Triassic age, covered by an unconformable veneer of Glacial and Recent sediments. The upper zones of the saltbeds are corroded by aggressive undersaturated groundwaters. It is thought that much of the solution of the rocksalt beds was the result of Glacial meltwaters, reactivated by the removal of the protective saturated brines by industrial pumping which, for example removed some 2.2 million m^3 of brine in 1948. The removal of the rocksalt has withdrawn support from the overlying Triassic breccias, Glacial and Recent sediments and their collapse has induced a suite of salt-karst land forms at the ground surface which include craters 200m in diameter, linear hollows 240m wide and 9km in length as well as areas over $2km^2$ in extent which have been generally lowered.

Precise Topographic Surveys

There is a well recognised need for precise topographic surveys and this is done by the Ordnance Survey of Great Britain. Local surveys are also made by surface users with specific interests. There is no substitute of such surveys where precise measurements are required. However such surveys have inherant limitations for they do not identify conditions between the observation points. Unless these points are closely spaced the overall nature of the terrain cannot be interpreted.

The Role of Photogeology in the Study of Rocksalt Subsidences

The nature of the terrain indicates the condition of the subsidence cycle at a given time. (Howell, Jenkins 1976). The evolution of the subsidence can only be recognised if changes in the morphology of the land surface can be observed, recorded, and compared during the passage of time. Photogeology, that is the skilled geological interpretation of photographs, thus forms an invaluable supplement to precise topography surveys. In combination, precise topographic surveys and photogeology best shows the evolution of salt subsidences, thereby throwing light upon the mechanisms of rocksalt subsidences, which in turn may assist in predicting future subsidences, and devising methods for combatting such subsidences.

Photogeology available of Cheshire Rocksalt Subsidences

The photogeology available falls conveniently into two groups.

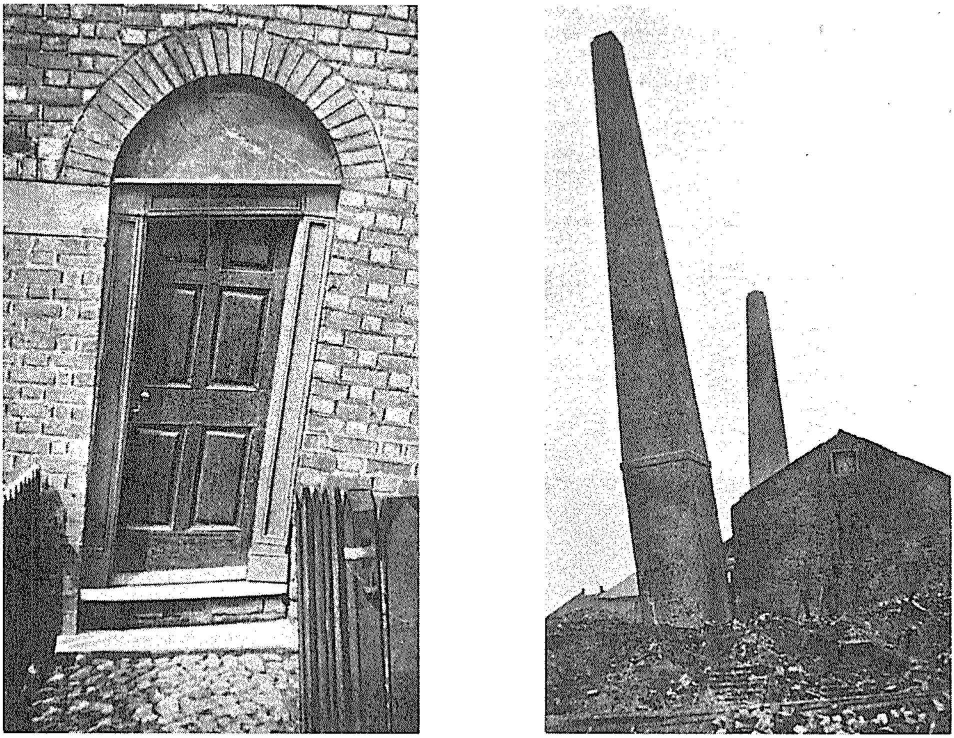
A. Those photographs not specifically taken for photogeology interpretation.

B. Those photographs taken specifically for photogeology interpretation.

A. Many photographs are available which were taken at ground level. These are mainly in black and white and have a general use in recording views of subsidences or collapses which were sufficiently noteworthy to photograph. Some were taken prior to 1900AD.(Fig 1)

Fig. 1 Crater, circa 1900. (Calvert).

The disadvantage of such photographs is they only record a very small fraction of the surface of the region, do not form a systematic record, and only usefully record the near ground. They however show well the sides of structures (Fig 2,3). Air photographs can be more informative



Figs. 2 and 3 Structural damage, circa 1900. (Calvert).

for the elevated viewpoint is generally advantageous. Many general oblique photographs have undoubtedly been taken of the region over many years. Unfortunately such photographs were not taken systematically, and are scattered throughout innumerable collections. The vast majority of such photographs were taken in black and white but the recent increase in the popularity of colour photographs for general use will probably reverse this balance in due time. Vertical air photographs are of especial interest, for they do not suffer foreshortening of the view and the image can be conveniently transposed onto maps or plans. Moreover, vertical air photographs are usually the result of a systematic overflight of considerable extent. In such air surveys the images are usually arranged to give a steroscopic view. The general effectiveness of vertical air surveys for many purposes other than photogeology has been well recognised in Cheshire and as a consequence prior to 1980 no less than four surveys had been made using large format black and white film. These surveys included a large proportion of the saltfield on these occasions. (Table 1)

These air surveys were taken at differing heights, in differing seasons and lighting conditions, and therefore do not provide information which is strictly comparable. Nevertheless they have captured an immense amount of information which shows the extension of land subsidence. (Fig 4)

Table 1. : Professional Air Surveys of parts of Cheshire Saltfield.

Prior to 1980 (Black/White)

Coverage	Scale	Date
Partial	1: 9840	1947
Partial	1: 62000	1969
Complete	1: 10000	1971-73
Partial	1: 25,000	1976

Fig. 4 Subsidence growth, 1960-73 (D.O.E. Hunting)

At U.M.I.S.T. some portions of these photographs have been converted into a mozaic of pixels on a grey tone scale and computer manipulated to enable direct comparisons to be made between photographs taken on different occasions. (C.M.C. Jones, Personal Communication)
This technique reduces the subjective nature of visual comparisons and has assisted not only in confirming extensions of the more obvious forms

of subsidence, but also in identifying subtle changes in the grey-tone reflectance of the terrain or overlying vegetation.

B. Photographs taken especially for Photogeology Interpretation.

For several years experiments have been taking place under my direction at U.M.I.S.T. to determine the optimum methods of taking photographs of the rocksalts subsidences of Cheshire. In this, due regard has been made to cost, for the author is of the opinion that frequent photographic surveys of adequate, but not necessarily the highest quality, may in combination have more value than less frequent photogaphic surveys of higher quality and cost. Indeed if this argument is taken to its limit, it will be appreciated that any photograph may be better than none. The aim in the studies has been to optimise the competing demands of technical excellence and cost. In this it is necessary to capture adequately the shape, terrain and texture of landforms representing the ground surface as well as the nature of the overlying vegetation, for the condition of the vegetation may be related to groundwater response of underlying geologial conditions for in the saltfield the interaction between the strata and groundwaters is a key feature in the mechanism of rocksalt subsidences. (Howell, Jenkins 1976, 1980., Jenkins 1983, Howell in press). Many photographs were taken on the ground simply to evaluate film emulsions. It became evident that 35mm cameras were capable of producing images of adequate quality, providing the minimum sized object to be recorded has sufficient angular size to overcome the texture of the image resulting from the grain of the film emulsion. The angular size of the image is influenced by the distance and size of the object, as well as the focal length of the lens. Vibration or movement of the camera during film exposure must independantly be within acceptable limits. Experiments were conducted with arrays of cameras, in some instances as many as five identical low cost 35mm format cameras were used to compare the simultaneous images captured by the various film and filter combinations. The impression formed was that natural colour, false colour infra-red and black/white panchromatic films were, in that order the most effective. Least effective were black/white Infra-red film and black/white film using inappropriate colour filters. Since pairs of images taken simultaneously produce stereoscopic effects, full colour stereoscopic images could be made using two cameras containing colour film. A comparable result could be obtained with cameras containing false colour infra-red film. It was also found that when the density of the images was not too dissimilar to an 'uncoloured' stereoscopic image could be derived from a single colour and single false colour image taken simultaneously from suitably separated cameras. (See Table 2).

Such ground experiments were used to record oblique views of ground subsidences and the overlying vegetation.

Their limitations were those already reviewed in respect of other oblique ground photographs. Some were superior in that natural colour is a more complete record than black and white photography. False colour images appeared to enhance contrasts, but it is by no means certain that on every occasion they were detecting features which could not also be percieved on natural colour images. However this effect is being evaluated by R. W. Gough (Personal Communication).

Table 2 : Potential Properties of Obliquely Directed Cameras

	Single Camera			Two Cameras		
Film Type	B/W	C	F.C	B/W	C	F.C.
Plan distorted	*	*	*	*	*	*
Stereoscopic				*	*	*
Texture	*	*	*	*	*	*
Natural Colour		*			*	
False Colour			*			*

Film Type : - B/W black - white
C colour
F.C. false colour

An intermediate stage was reached in the experiments when a helicopter was used as a platform for oblique photographs. Although the helicopter can hover the experience was that this advantage was more than offset by vibrations which gave a greater angular movement during the exposure of the photograph than that in a light aircraft. The corresponding cost of helicopter hire is several times that of the single engine light aircraft in (Cessna) in which subsequent experiments were made. (Fig 5)

In the light aircraft available, oblique photographs could be hand held either through the unopened windows or through open windows. The presence of the perspex window had a variable effect on the photographs. Sometimes the images appeared unimpaired. Other times flair or distortion was noted. However on balance the comfort of having a closed window seemed to tend towards a better overall collection of photographs. The limitation of the window view was that in practice only two cameras could be used simultaneously. The choice appeared to be towards either two containing natural colour, or two false colour, or one of each. Although in general 35mm format oblique photographs could be obtained with a fair rate of success at very low cost, the limitations of occasional flair, distortion and above all the obliqueness of the view indicated that there substantial improvement might result from vertically mounted cameras. At that time the Cheshire

County Council were considering commissioning a professional air surveyof the County in natural colour, which encompassed the rocksalt subsidence districts. Ongoing scientific collaboration between the County and U.M.I.S.T., supported by the Science Engineering Research Council (S.E.R.C.) led to an extension of the scope of the survey in which vertically directed false colour infra-red and natural colour photographs were to be taken simultaneously with sufficient overlap

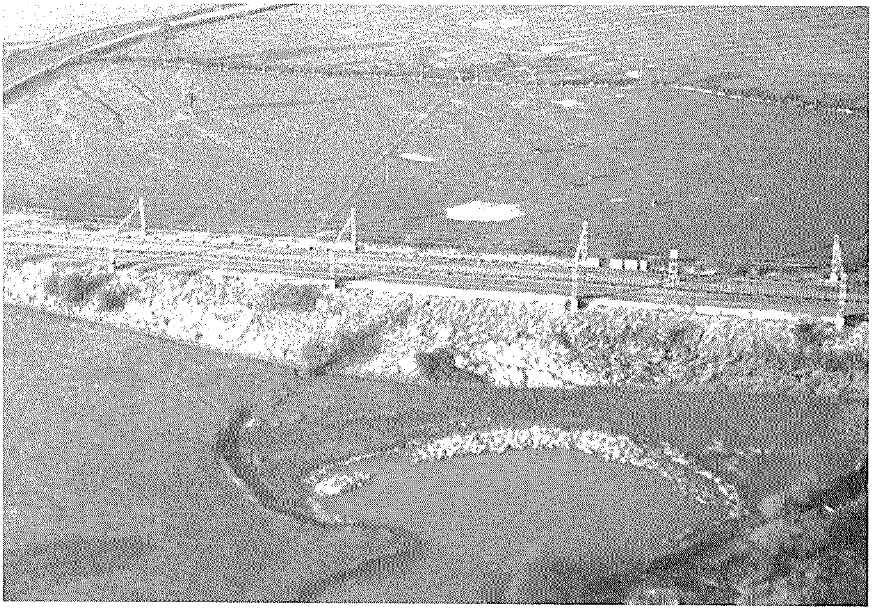


Fig. 5 Crater and scars (top left) Howell

along the flightpath to produce stereoscopic natural colour and stereoscopic false colour images. In this manner not only could stereoscopic images in the two emulsions be obtained but also an identical false colour and natural colour image could be available for comparison. This air survey contract, financed by the Cheshire County Council was awarded to Airviews of Manchester. The survey, using 6 x 6 cm format was started in the summer of 1983 but various combinations of adverse weather conditions such as haze, cloud shadow and rain greatly delayed the survey and do show a practical limitation to false colour infra-red photography in wet temperature climates.

Nevertheless, when obtained the first test results demonstrate the superiority of vertically directed photographs over oblique ones, and the superiority of matching natural colour, and false colour photographs over natural colour, or black and white photographs. (See Table 3, and Fig 6)

Photogeology of the Cheshire Salt Subsidences

The land surface associated with the Cheshire Salt subsidences contains a suite of landforms (Howell, Jenkins 1976).

In order for these to be recognised on the photograph they must produce an acceptable image. The characteristics of the image required differs with the various landforms. Whether or not the landform can be identified depends upon the presence in the photography of the key characteristics. It will be evident that the image size must be large

Table 3 : Potential Properties of Vertically Directed Cameras

	Single Camera			Two Cameras		
Film Type	B/W	C	F.C.	B/W	C	F.C.
Plan View	*	*	*	*	*	*
Stereoscopic	by movement			*	*	*
Texture	*	*	*	*	*	*
Natural Colour		*			*	
False Colour			*			*

Film Type : - B/W black - white
C colour
F.C. false colour

Table 4 : Landforms - Typical Dimensions

Landform	Size in Metres W	L	D	Photograpic success from air
Small crater	10	circ.	5*	moderate
larger crater	100	circ.	30*	frequent
cracks	.1	5	0	rare
terraces	1	10	1	infrequent
linear hollow	100	1000	5-10*	frequent
Regional lowering	1000	1000	10*	frequent but not diagnostic
Potential subsidence SITES				not proven but may have potential

W = Width L = Length D = Depth * tend to flood/

NOTE. 1. All features can be photographed from the ground if near enough.

2. Currently available satellite images can only resolve largest features.

Fig 6. Major crater, sediment infill (Airviews) (Cheshire County Council)

enough to be captured on the film and this depends upon height, camera optics, film type and movement or vibration. See Table 4

The impression of differences in elevation of the ground surface may be captured in a single photograph by the effects of shadows. Where stereoscopic images are available shadows are less important, indeed, they can be unhelpful for detail may be lost in either the shade or the highlight of the image. Texture of the ground, or of the overlying vegetation may assist in interpretation. Colour and false colour infra-red photographs capture more information than black and white photographs.

The field condition of these can be subdivided threefold, for some are dry, others are associated with damp ground and some flood. In general the larger depressions such as large craters, linear hollows and areas of regional lowing tend to flood. However the presence or absence of water is variable depending upon the weather and local hydrogeological constraints. Reference to table 4 shows that cracks which have not dislocated to form terraces (for example around craters or adjacent to linear hollows) are particularly difficult to identify for they are not only small in size but they are not associated with a change in ground elevation. Thus they do not cast a shadow and are not revealed by stereoscopic examination. Vertical dislocations associated with some cracks cause a displacement of the land surface and in some circumstances the presence of a shadow will be noted in single photographs, and as a change in elevation in stereoscopic photographs. Small craters may cast a shadow and providing they form an image of adequate size they usually are identified by their shadow in single photographs or by their shape in stereoscopic viewing. Colour and false-colour infra red images tend to capture subtle changes in reflectance, texture, or colour of these small features or their vegetation. Large craters and linear hollows are usually easily identified on photographs for they cast shadows and tend to be water filled or damp. Surprisingly perhaps, areas of regional lowering are less easy to recongise with certainty for the effect is widespread, and slope changes are not distinctive, although regional lowering is often associated with poor drainage, river diversions and lakes ("meres"). The overall nature of the dimensions of the larger subsidence landforms is that for the most part they can be readily recorded photographically. Only cracks, some terraces and small craters prove to be especially difficult to identify. The problem revolves round the limitations of resolution and contrast of the image. Although on many occasions photography can identify subsidence features, their absence on a photograph does not mean that they do not exist. For this reason photographs must be used with caution. Often in wet weather small hollows fill with water and can be identified. Subsequent photographs may not show them; not necessarily because they have been infilled, but because a low relief dry or damp hollow has less contrast than a waterfilled pool. (Gavin Tudor, Personal Communication)

Some infilling of hollows does take place, either by man's activities, sedimentation or the encroachment of vegetation. These infillings may be areas of potential problems. Although the new topography may not indicate the former hollow, vegetation tends to exhibit a localized contast and colour, and false colour photographs may have the greatest potential to detect such sites. The development of craters is particularly important in the subsidence cycle, for the craters indicate, often for the first time that the rocksalt is dissolving. Since craters can be particularly sudden and destructive it would be of great value to be able to identify the location of future craters. The rapidity of the crater development is such that it cannot be accounted for by the rate of solution of rocksalt. A possible explanation, demonstrated by modelling crater development in the U.M.I.S.T. 100 G centrifuge (Howell and Jenkins 1984) is that the strata overlying the cavities in the rocksalt may move into the salt cavities for long periods without any movement appearing as subsidence at the ground surface. This is accomplished by an increase in the general void space in these materials until a critical minimum density is reached. If this is indeed the case then changes in the groundwater regime and in the health of the overlying vegetation may accompany such changes for considerable periods of time prior to the appearance of the crater. However, to date the effect has not been confirmed in the saltfield. The resolution of currently available satellite images makes them unsuitable for the detection of all but the largest of subsidence landforms. However remote sensing supplemented by radiations outside the visible spectrum is likely to become a useful tool when adequate the resolution is available.

Conclusion

Many photographic techniques have been used to record the rocksalt subsidences of Cheshire and these significantly supplement accurate topographic surveys. Oblique ground based photographs can be of use despite their qualitative nature. The optimum cost-quality value appears to be reached in the use of two 6 x 6 cm synchronous cameras vertically mounted in a light air craft and using natural colour and false colour infra-red film.

Acknowledgements

Thanks to N.E.R.C., S.E.R.C. S.R.C., Cheshire County Council, British Rail and U.M.I.S.T. for funding and encouragement.

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TOPOGRAPHIC CHANGES IN THE TRAVALE-RADICONDOLI GEOTHERMAL FIELD DURING THE FIRST TEN YEARS OF EXPLOITATION[1]

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[1] Work conducted as part of the CNR-ENEL collaboration agreement

Abstract

Between 1973 and 1983 about 220 km of precision levelling were carried out during 12 separate surveys in the Travale-Radicondoli area (Tuscany,Italy).This area includes a geothermal field of about 2 km radius, with producing and reinjection wells. The results of these observations reveals a maximum subsidence in the central part of this area at an average rate of 25 mm/yr for the last ten years.Three series of horizontal distance measurements were carried out (in 1980,1981 and 1982)to monitor variations in the coordinates of the benchmarks of a horizontal control net.The variations noted in this time interval range between 13 and 36 mm, exceeding the semi-axis values of their error ellipses. The tentative correlation of these data with field production data indicates the possibility of interpreting the variations noted in terms of a pressure drop in the exploited reservoir.

Survey of vertical changes

A network of elevation benchmarks was established in the Travale geothermal area and the first measurements made in June 1973 using precision geometric levelling. The local geothermal power-plant began operations one month later, fed by the first well drilled in this "new field", T 22 (Burgassi et al.,1975).

Vertical changes (Δh) in the area throughout the various phases of development and exploitation of this field were monitored by 12 different surveys on the network over a period of 10 years(Geri et al.,1981,1983). At present the levelling network of 180 benchmarks covers about 40 km; 70% of the lines run along rough country roads, winding or steeply sloping lanes, through woods and thick vegetation. The kilometric m.s.e. of the first 11 levellings was $\pm$ 1.47 mm (Geri et al.,1983).

Statistical analysis of the results (Geri et al.,1983)showed that the changes in elevation, between two levellings in different periods, of the benchmarks within the exploited area with respect to an external reference benchmark,can be estimated with a m.s.e. ranging between 3 and 6 mm. Vertical changes of this order of magnitude, however, can be detected when they represent a 'tendency' characteristic of all, or nearly all, the benchmarks in the study area.

Figures 1,2 and 3 show subsidence trend with time, especially the roughly elliptical depression whose longest axis trends NE-SW.

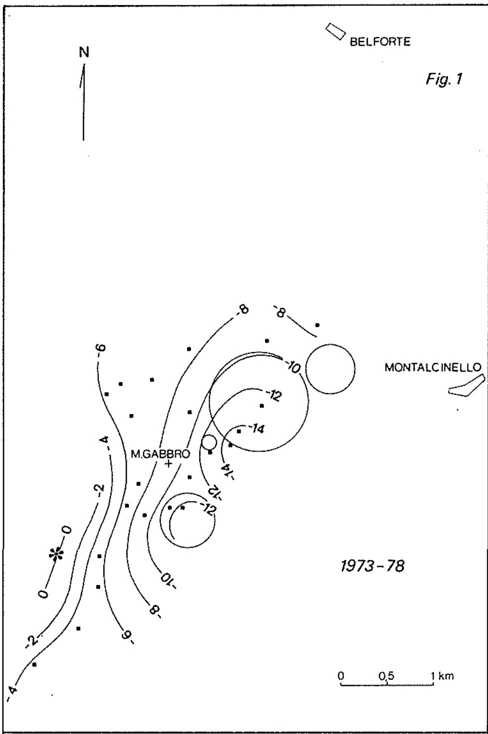

Fig. 1

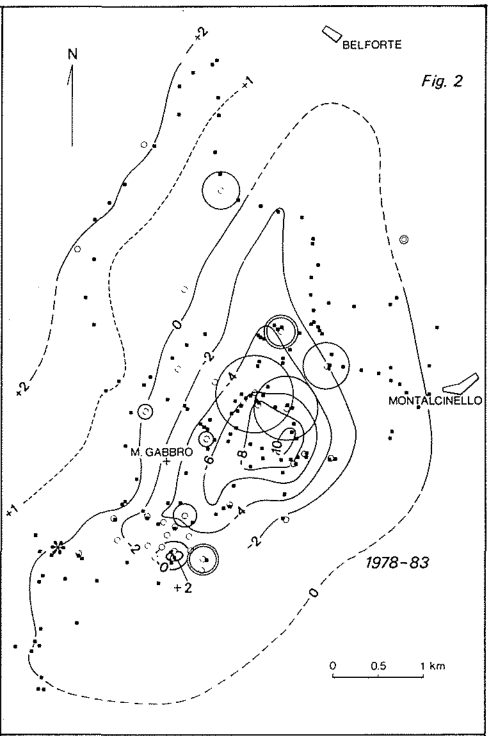

Fig. 2

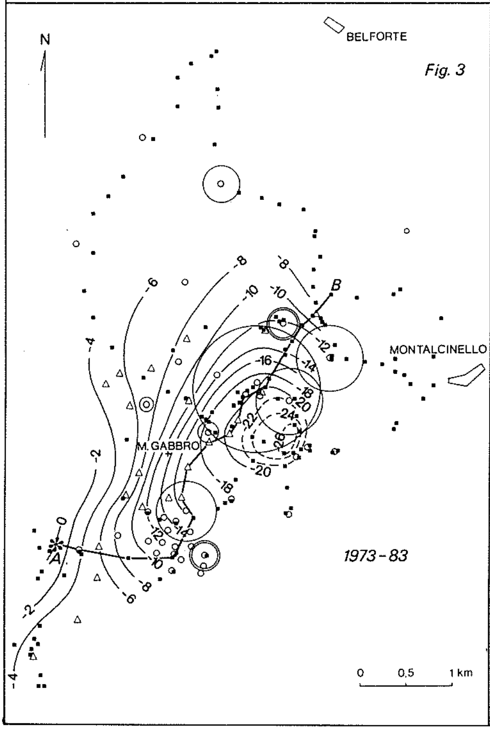

Fig. 3

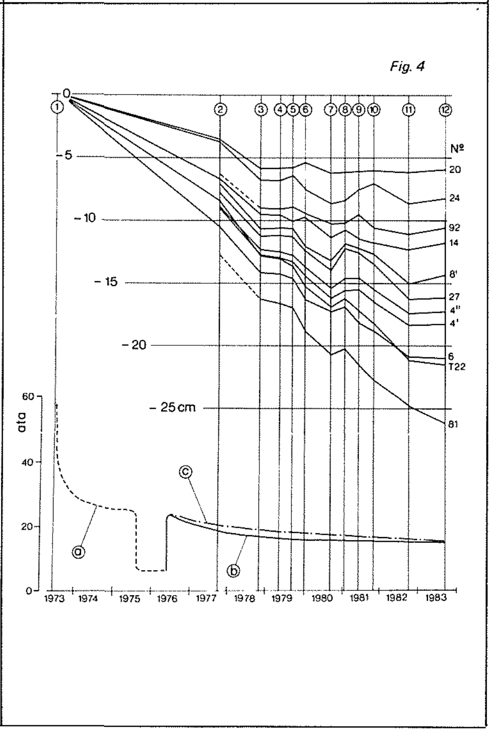

Fig. 4

This trend can be correlated with exploitation of the geothermal field, and also with the geological-structural elements that gave origin to the latter.The most important factor seems to be the main fracture and fault systems that created preferential permeability lines, as revealed by the magnetotelluric and aerophotogrammetric surveys (Celati et al.,1973) and seismic surveys (Batini et al.,1978).

The subsidence contours are quite regular and uniformly spaced, and seem to be unaffected either by surface geology or the geotechnical characteristics of the benchmarks (type of structure, foundation,etc.).

The depression, including its marginal sectors, is wider than the area covered so far by the network; since the depression also affects the outcrops of Mesozoic carbonates about 3 km south-west of the field, it is probable that the stable reference benchmark (see * in Figs.1,2 and 3 and A in Fig.6) has moved with respect to the areas further from the geothermal field, where the highest subsidence rates were observed. A positive change in elevation can, for example, be noted on the north-western margin of Fig.2.

Subsidence rate was higher in the period 1973-78 (Fig.1) than in the period 1978-83 (Fig.2), yet the levelling network in the first 5 years did not cover the area that showed the highest subsidence rate in the 78-83 period. This is also evident in Fig.4, which shows the vertical change with time of some benchmarks relative to their zero elevation of 1973. Note that the benchmarks in the marginal areas with lower subsidence values (Nos.20,2,24) registered almost zero subsidence rates in the 78-83 period.

Survey of horizontal changes

Horizontal ground movement was monitored by means of a horizontal network with 7 bases whose coordinates were referred to a local system. The lines were observed in 1980, 1981 and 1982, using medium-range electronic distance measuring equipment (AGA), with a precision better than 20 mm per 3 km of line length.

Figure 5 shows the horizontal movement vectors and relative error ellipses for the period 1980-1982, holding base B as conventional reference point.

The relative movements range between 13 and 36 mm and indicate that the area covered by the network is gradually shrinking.

Figures 1,2 and 3: Ground vertical changes,in cm,during the 1973-78, 1978-83 and 1973-83 periods respectively.All elevation changes were calculated by holding benchmark no.25 (*)stable during the relevant period. Circles indicate sites of exploited wells and total production during the period in question. Double circles indicate sites of reinjection wells and the quantity of water reinjected. A-B: see cross-section in Fig.6.

Figure 4:Top:subsidence of selected benchmarks within the Travale-Radicondoli geothermal field.Bottom:wellhead pressure behaviour, in different conditions, in the central part of the 'new field' during exploitation. a) T22 producing well; b) T22 shut-in well; c) T23 shut-in well.

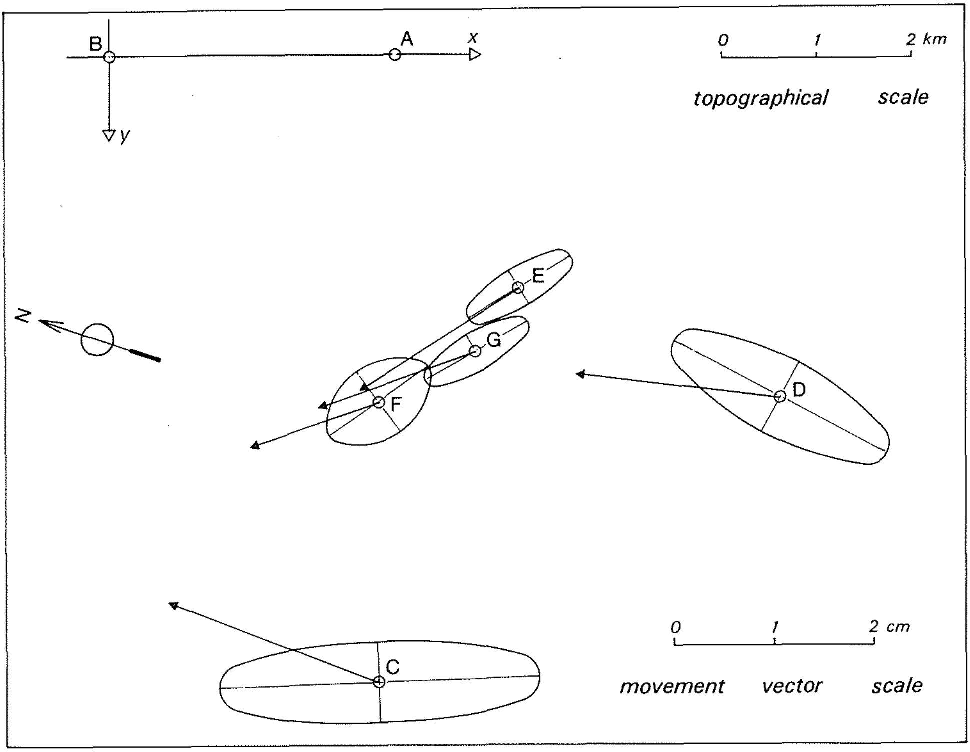

Figure 5. Horizontal change vectors and relative error ellipses.

Structural and hydrological remarks on the Travale-Radicondoli field

The main characteristics of the Travale-Radicondoli field (Cataldi et al.,1970; Burgassi et al., 1975) can be observed moving southwest--northeastwards along the cross-section in Fig.6:

area a), where the outcropping Mesozoic carbonate formations of the Tuscan Series,which form the main geothermal reservoir, have no cover and absorb the cold meteoric waters; because of favourable permeability and structural conditions in this area these waters are able to infiltrate to depth, especially into area b), immediately northeast of area a);

area b), termed the 'old field', is characterized by an impermeable cover formation of an average thickness of a few hundred metres, and by a very permeable reservoir in which the shallow cold waters from area a) circulate and mix with the hot fluids rising from depth and from adjacent area c);

area c), termed the 'new field', where the carbonate reservoir is further lowered by an important NW-SE trending fracture system.This reservoir is buried beneath impermeable cover formations varying in thickness between 600 and 1800 m,and produces high-temperature and high-pressure fluids with, at times, a CO_2 content of more than 50%.

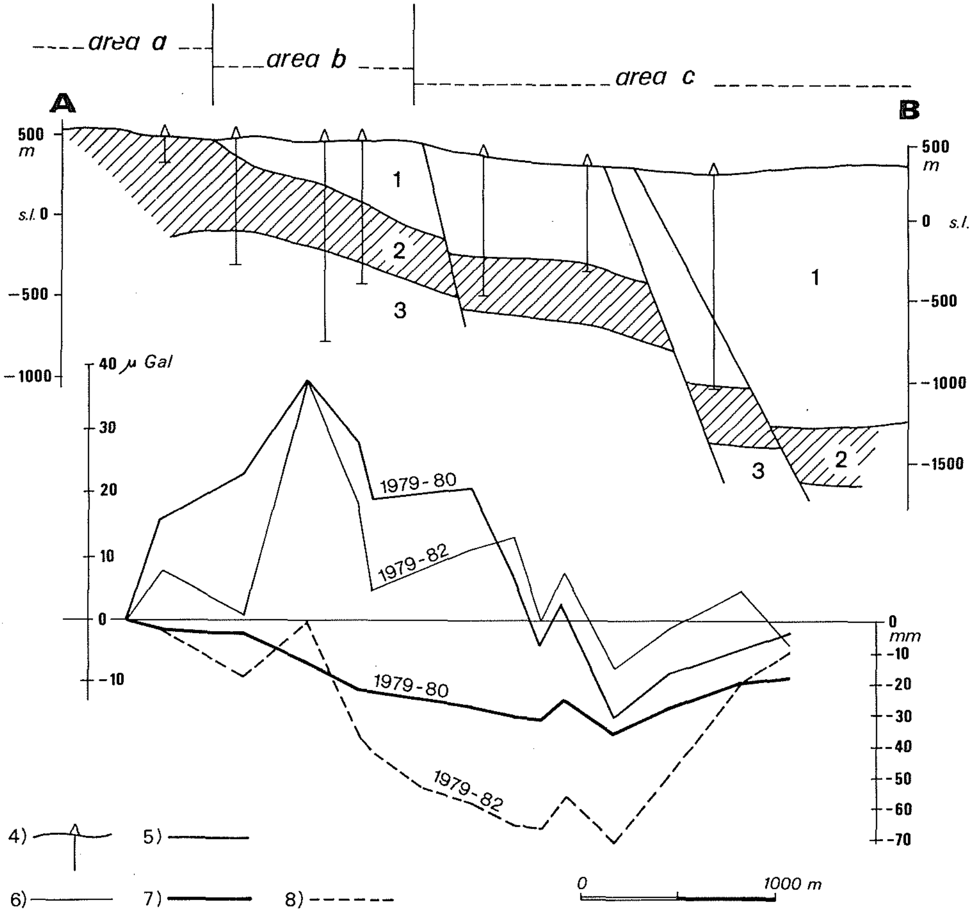

Figure 6. A-B schematic cross-section of Fig.3. a= area of permeable outcrops; b= 'old field' area; c= 'new field' area. 1) impermeable cover complex; 2) main reservoir; 3) metamorphic complex; 4) drilled wells; 5) gravity variations corrected for topographic changes (freeair correction) for the period March 1979-September 1980; 6) idem for the period March 1979-November 1982; 7) ground vertical movement (November 1978-October 1980); 8) ground vertical movement (November 1978-October 1982).

The formations of the underlying metamorphic basement are also directly involved in the production of geothermal fluids.

The liquid phase predominates in the 'old field' reservoir, whereas the gas phase (steam + CO_2) predominates in the 'new field'.

A few years of relatively intense exploitation in the 'old field' during the 1950s led to such a large influx of cold shallow waters from the permeable outcrops in area a) that industrial exploitation had to be abandoned in 1962 owing to low fluid temperatures and gradual blockage of the wells through scaling (Cataldi et al.,1970).

Some wells in the 'old field' were later cleaned and re-opened, and left to discharge a total of 50-60 t/h of steam and water into the atmosphere until 1978-79, when they were again shut-in definitively.

After the first shut-in during 1964-68, and that of 1978-79, the piezometric level rose in the 'old field' and the drainage cone gradually filled. The positive g variations observed in the 'old field' between 1979 and 1980 (Geri et al.,1982; Geri et al.,1984,in prep.) can easily be ascribed to a density increase resulting from water saturation of the upper layers of the porous reservoir rocks (see also Fig.6, area b)).

The lower part of Fig.7 shows the few data available on the trend of the piezometric levels in some non-productive wells of the 'old field' during the last ten years, compared to the elevation changes in the topographic benchmarks in the same area.After the final shut-in of the wells (1979) and concomitant with the rise in piezometric levels, one group of benchmarks (Nos.90, 91,123,124;see upper part of Fig.7) exhibited positive vertical changes, as opposed to their behaviour in earlier years and the behaviour of the benchmarks in other zones of the field (Fig.4).

In Fig.2 the 'old field' area again exhibits an unusual positive Δh anomaly of more than 2 cm,observed over the last 5 years.

These observations accord with the hypothesis forwarded by Cataldi et al.(1970) for the geothermal system in the 'old' Travale field; according to this model the influx of cold meteoric waters from the permeable outcrops of area a) may increase proportionally with the amount of fluid extracted from the 'old field' area b), and decrease whenever production stops and the piezometric levels almost return to their original values. Note, however, that the flow of fresh,shallow waters from the 'old' to the 'new' field is now higher, due to the rise of the piezometric level in the 'old field', and the decrease in reservoir pressure in the 'new field' resulting from 10 years of exploitation.

The latter hypothesis also seems to be corroborated by the areal trend of subsidence (Figs.1,2,3), which reveals the existence of a preferential flow-path extending from the 'new field' through the 'old field' to the permeable outcrops.

Subsidence, production and reinjection

Fluid production in the 'new field' has been evaluated at about 11.5 x 10^6 tons for the period 1973-78, and 13.2 x 10^6 tons for the period 1978-83, totalling 24.7 x 10^6 tons; a further 3.3 x 10^6 tons of water and steam were also produced in the 'old field' in the period 1973-79, bringing the total to about 28 x 10^6 tons.Reinjection tests began in 1979 with 100,000 tons/yr, and increased to 450,000 tons in 1982, which corresponds to about one-sixth of the mass of fluids extracted that year. A total of about 2 x 10^6 tons has been reinjected so far.Subtracting the 2 x 10^6 tons of water reinjected back into the field, we may deduce that about 26 x 10^6 tons of fluid were extracted from the Radicondoli-Travale field between 1973 and 1983.

Figures 1,2 and 3 reveal that the production areas correspond

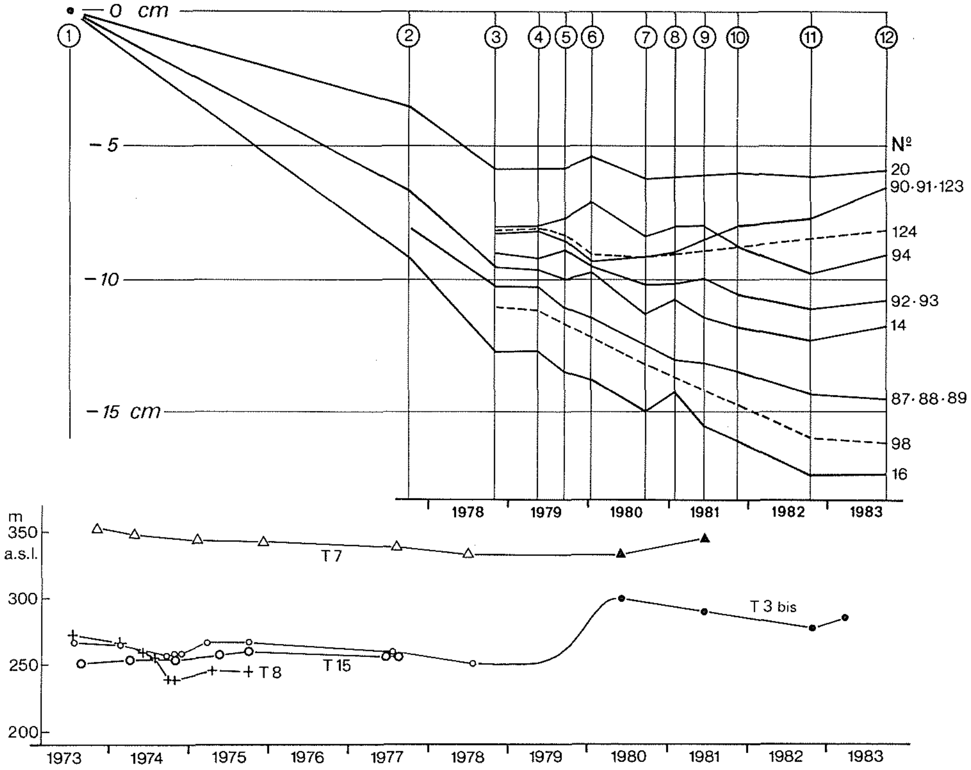

Figure 7.Upper part: Elevation changes in selected benchmarks around and in the 'old field' area.Lower part: 'old field' piezometric trend in some non-productive wells. Empty and black symbols represent before and after shut-in of the productive wells respectively. The wells that discharged small quantities of water and steam into the atmosphere were shut-in between autumn 1978 and autumn 1979.

almost exactly to the areas with the highest subsidence rate, although their centres do not coincide.Note also that reinjection is a negligible factor as far as subsidence is concerned.

The depression which has formed over the last 10 years as a consequence of subsidence can be described as a cone, with an irregular base covering an area of about 28 km^2, and a maximum depth of 26 cm. The volume of the cone is 2.4×10^6 m^3, corresponding to about 9% of the volume of extracted fluids, considered as water only.

Figure 4 shows the subsidence observed in some representative benchmarks and the pressure variations recorded at wellhead in two productive, but unexploited wells sited very near producing wells in the centre of the 'new field'.This diagram illustrates the pressure trend in the central part of the productive area during fluid extraction from a vapour-dominated reservoir.

A pressure drop of about 4 ata in wells T22 and T23 in the period 1977-82, corresponding to a drop of about 40m in the piezometric level, was accompanied by a subsidence of about 10.5 cm in the same area; subsidence rate reached the same value in only 4 years during the first production period (1973-77), but with an accompanying pressure decrease of the order of 20 ata (10 of which occurred during the first year).

Conclusions

The following observations can be drawn from the results of the surveys described above. These work hypotheses must be verified in future studies of this area:

1) subsidence is probably a consequence of compaction of the reservoir resulting from a pressure decrease within the rock pores and fractures; the depression is the result of fluid extraction;
2) since the reservoir consists of porous, fractured carbonate rocks and more compact, phyllitic-quartzitic rocks of the underlying metamorphic basement, it is not easily affected by plastic deformations.Small-scale deformations may,however, be triggered or enhanced by widespread micro- and macro-fracture systems;
3) compaction probably also affects the so-called cover formations (flysch and recent, poorly consolidated clastic deposits) despite their low permeability; indeed the small amount of water contained in these rocks may vary considerably because of the pressure drop in the underlying reservoir, thus leading to volume changes in the cover, especially in the clayey sectors.
4) only a fraction of the fluids extracted are replaced by other fluids coming from adjacent areas and deeper zones, or through reinjection;
5) although the data available do not cover all the area affected by the depression, the latter does seem to have gradually extended to the Mesozoic carbonate outcrops containing the reference benchmark; these outcrops were considered more stable and unlikely to be affected by subsidence, because they act as recharge areas for the meteoric waters.

   Should future surveys confirm that subsidence is affecting the carbonate outcrops, then we could hypothesize that the values reached throughout the field are higher than those observed so far.

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CRITERIA FOR EVALUATING SOIL COMPRESSIBILITY AND PERMEABILITY IN AREAS SUBJECTED TO SUBSIDENCE DUE TO HYDROCARBON RESERVOIRS PRODUCTION.

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Abstract

After reporting the causes of error when estimating in the laboratory the compressibility and permeability of unconsolidated clastic rocks (clays, silts and sands), the importance of determining, by means of a preliminary investigation, the areas where the error mostly influences the results is ponted out.

The different causes of error are then quantified on the basis of the tests performed on the materials in the Po Valley.

A range of probable compressibility values in such area are then reported, for soils from 500 m to about 3,000 m in depth, stressing the limits arising when investigating a specific reservoir.

Introduction

The important developments in the modeling technique applied to the study of the subsidence should be accompanied, in order to obtain more significant global results, by an analogous improvement of the techniques for determining in the laboratory the physical and mechanical parameters characterizing the soil.

Yet the results obtained in this domain do not appear to be fully satisfactory owing to both duration and costs of the experimental studies and, above all, to the difficulties of these studies in the case of unconsolidated materials, such as sands and clays.

Such difficulties are due to:

- the small number of the samples drilled, taking into account the considerable heterogeneities characterizing the soil;
- the impossibility of obtaining completely undisturbed samples actually representative of the medium;
- the difficulty in faithfully reproducing in the laboratory the phenomenon occurring in situ.

When estimating the parameters characterizing the medium, it is almost impossible to achieve the same accuracy obtained by the simulation technique. On the other hand, the technician should find somehow the answer to problems which often involve important issues concerning the protection of the environment.

The line to follow should be that of trying to determine such parameters according to more methodologies, associating the laboratory estimates with the measures taken in situ and evaluating the reliability of the results obtained on the basis of the history of the phenomenon.

Parameters which mostly influence the phenomenon of subsidence

Given the difficulties of the experimental tests, the investigations should be concentrated on the objctive detection of the parameters which mostly influence the subsidence.

Leaving aside therefore the study of the effects of size and depth of the reservoir as well as the extent of the decrease in pore pressure- effects which were already stressed by Geertsma (1973) - the parameters permeability and mechanical characteristics of the rocks are here considered, also referring to some interesting results obtained by Van Opstal (1974), Brighenti (1976) and Borsetto and Carradori (1983).

As far as permeability is concerned, it should be determined in the reservoir, in the adjacent aquifer, in the overlaying rocks and in the bedrocks, i.e. in those areas where the exploitation of the reservoir may cause a decrease in pore pressure.

Special attention should be paid to adjacent rocks, which are usually characterized by an extremely low permeability. For this reason, they shold be concerned by the phenomenon during the exploitation time and the subsequent re-pressurization for variable degrees of thickness, from few centimeters to few meters.

Obviously, this is true when these rocks are not drained by intercalated thin layers with higher permeability. It can therefore be observed that, in these cases, the determination of the permeability on a large scale by means of investigations in situ is more interesting than the estimate of the local permeability on samples in the laboratory.

As to the mechanical parameters which connect stress and strain, beyond of course the parameters of the layers subject to a decrease in pore pressure, the parameters concerning all the area subject to remarkable stress variations influence the phenomenon of soil subsidence.

Some results regarding a disk reservoir are reported in the following. Though they are not exhaustive, they may provide the researcher with useful information; more precisely, in the case of a elastic medium:

1. the influence of the rigidity of the basement and its distance from the reservoir (Fig. 1a and 1b);
2. the influence of the overlaying layers (Fig. 1c).

A thorough examination of such results points out that it is very important to accurately determine the possible presence of a rigid basement, its characteristics and distance from the reservoir. As for as the mechanical characteristics of the soil overlaying the reservoir are concerned, their influence on soil sinking is remarkable. It was estimeed, however, that an outline of their trend, without dwelling on the local variations, would have sufficed, as can be seen also in the work by Brighenti (1976).

Always in the case of a elastic medium, it was found that the influence of the variation of the Poisson's ratio is negligible for $\nu = 0.20 \div 0.30$.

Obviously, a complete parametric study of this topic would be very useful to provide more accurate information.

Laboratory experiments

The main causes of the differences between the results obtained in the laboratory and in situ are due to:

a. the impossibility of obtaining undisturbed samples from loose rocks.

Also the use of sophisticated core barrels (such as the rubber sleeve core barrel introduced by Christensen before the Sixties) does not completely avoid the disturbance due to the mud, the constricting force of the rubber sleeve and the expansion of the interstitial fluids passing from the reservoir to the surface. Further disturbances may be due to the operations carried out in situ and in the laboratory;

b. the choice of the stress system to be applied in the laboratory. It is usually hypothesized that soil in situ is subject to the vertical lithostatic load and to the pore pressure and that it cannot deform laterally. Such schematization, often acceptable, can be reproducedin the laboratory either by a triaxial cell (imposing the condition of zero lateral deformation) or an oedometer. Many experimenters, however, to simplify the test, apply a uniform stress system and pass from the results thus obtained to those requested by using formulae valid for elastic materials;

c. the very high speed with which the stress field varies in the laboratory with respect to the reservoir. This leads to neglect most delayed strains and probably caused the grains to break more.

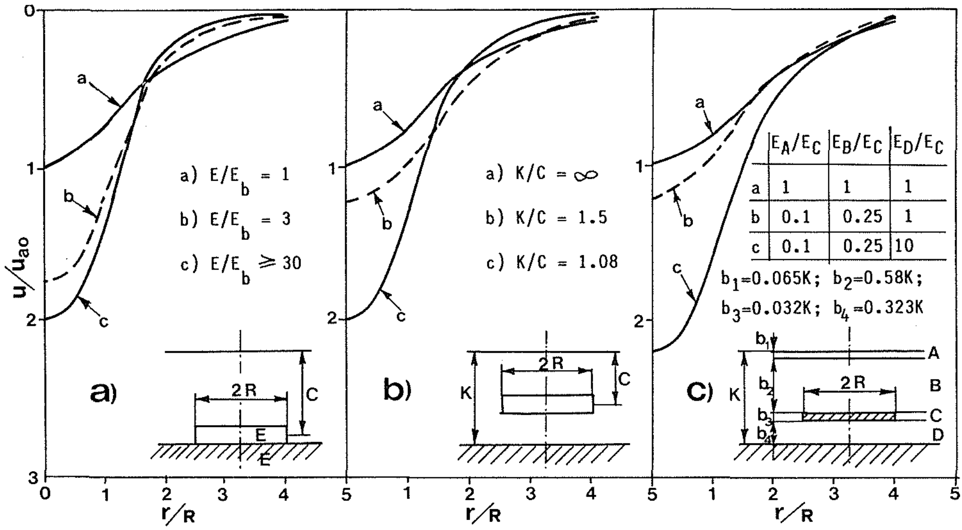

FIG. 1 a) Effect on subsidence of a stiffer base rock (Van Opstal 1974) C/R = 1.6; $\nu = 0.25$

b) Effect of a undeformable base rock at various depths (Van Opstal 1974) C/R = 1.6; $\nu = 0.25$

c) Effect of overlaying layers (Borsetto and Carradori 1983) R = 0.39K; $\nu = 0.2$

The quantities which are usually measured in the laboratory depending on the stresses applied are: the porosity n, the permeability k, the Poisson's ratio ν and the coefficient of compressibility.

The Authors have defined several coefficients of compressibility, i.e.:

the rock bulk compressibility:

$$C_b = - \frac{1}{V_b} \left(\frac{\delta V_b}{\delta \bar{\sigma}}\right)_{p,\ T}; \qquad (1)$$

the pore compressibility:

$$C_p = - \frac{1}{V_p} \left(\frac{\delta V_p}{\delta \bar{\sigma}}\right)_{p,\ T}; \qquad (2)$$

and the uniaxial compaction coefficient:

$$C_m = - \frac{1}{z} \left(\frac{\delta z}{\delta \sigma_z}\right)_{p,\ T,\ \varepsilon_x = \varepsilon_y = 0}; \qquad (3)$$

where V_b and V_p are the rock bulk volume and the pore volume respectively, z its hight, p the pore pressure and $\bar{\sigma} = (\sigma_x + \sigma_y + \sigma_z)/3$.

If the rock matrix compressibility is neglected, we have

$$C_b = n\ C_p\ . \qquad (4)$$

In the case of an isotropic elastic medium only, where E and ν are the Young's modulus and the Poisson's ratio respectively, Geertsma (1957) showed that:

$$C_b = \frac{3}{E} \ (\ 1 - 2\nu\ ), \qquad (5)$$

and

$$C_m = \frac{(\ 1 + \nu\ )}{3\ (1 - \nu)}\ C_b . \qquad (6)$$

He also showed that the variation of the medium's volume depends on $\bar{\sigma}$ only and not on the deviatory part of the stress tensor.

As already mentioned, the laboratory tests are performed by means of a hydrostatic cell, a triaxial cell or a uniaxial cell (oedometer) connected to a precision permeameter (see the works by Brighenti (1967), Teeuw (1971) and Brighenti and Fabbri (1982) for their description and test modalities).

On the basis of the tests carried out on about eighty disturbed and undisturbed samples taken from gas reservoirs in the Po Valley (sands, silts and clays) during the period 1962-1983, we estimeed it useful to illustrate some of the most common causes of uncertainty in laboratory estimates, and more precisely:

1. influence of the deviatory part of the stress tensor, the mean effective stress $\bar{\sigma}_e$ being equal. The comparison was made by considering different materials and different stress systems. Fig. 2 shows the case of three materials: a consolidated sandstone (a), a weakly cemented sand (b) and a loose sand (c) subject to uniform stress and to $\sigma_{ex} = \sigma_{ey} = \sigma_{ez}/3$ $\bar{\sigma}_e$ being equal. The results show that, as the behaviour of the materials is no more elastic, the values of C_b differ increasingly. The (6) is

therefore valid in the elastic case only. The differences between the two values amount to 30% on average in the case of a loose sand;

2. disturbed samples drilled by means of the rubber sleeve core barrel. Alla the samples tested were examined and deformed stratification planes were very often recorded (Fig. 3). All the sand samples and most clay samples did not show the expected variations of the trend of the curves ε_z - σ_{ez} next to the preconsolidation stress. This also shows that the samples are disturbed.
   In order to quantify the maximum effect of the disturbance in deep samples (drilled from more than 500 m in depth), a comparison was made between the results of some oedometric tests carried out on undisturbed and remoulded samples. Fig. 4 and 5 show the variations of C_m and k respectively in the case of sand and clay samples.
   It should be noted that, while the permeability has considerably greater changed, C_m has varied by 100% at the most. It is also possible to see that by increasing the applied stress and therefore the depth, the effect of the disturbance decreases as far as compressibility is concerned;

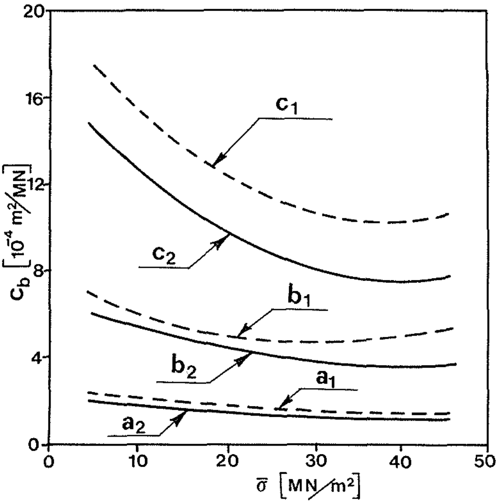

FIG. 2 Bulk compressibility: a - consolidated sandstone, b-weakly cemented sand, c - loose sand
1 - $\sigma_{ex}=\sigma_{ey}=\sigma_{ez}$;
2 - $\sigma_{ex}=\sigma_{ey}=\sigma_{ez}/3$

3. influence of repeated load cycles. The loose materials examined show a mainly irreversible behaviour. However, if they are subject to repeated load and unload cycles, they tend to behave in an elastic manner (Fig. 6). The compressibility and permeability obtained in the I and II load cycle vary remarkably (see, for example, C_m in Fig. 7).
   The problem is to state which values should be assumed as the most probable ones. If the sample were perfectly representative, one would agree with Newman (1973) and Mattex (1975), who suggested the values corresponding to the first cycle.
   The experience has shown, however, that such values are too high considering that the sample is somehow disturbed. Teeuw (1973) suggested that the mean value between the first and the second cycle should be assumed. While it is agreed that the value to assume should be intermediate between the two cycles, it is deemed that its evaluation should be based on the degree of disturbance which may be evaluated by comparing the characteristics of the material measured in the laboratory and in situ;

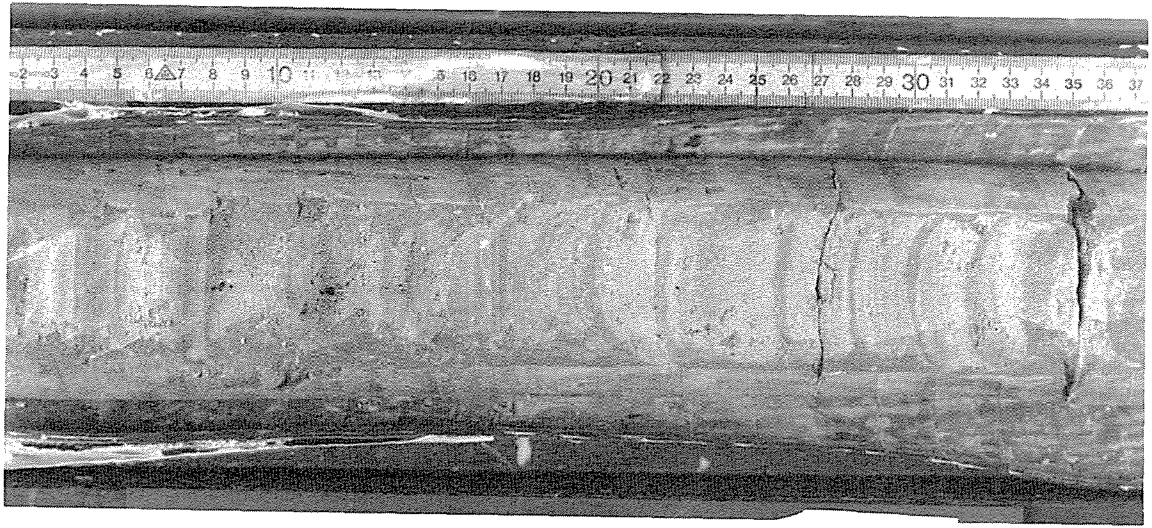

FIG. 3 Core drilled by means of the rubber sleeve core barrels

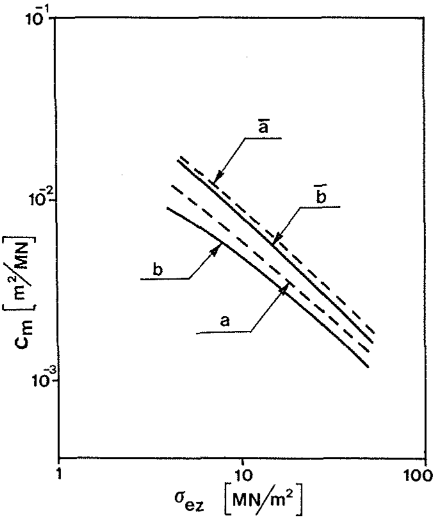

FIG. 4 C_m versus σ_{ez}: a - undisturbed sand, $\bar{a}$ - remoulded sand, b - undisturbed clay, $\bar{b}$ - remoulded clay

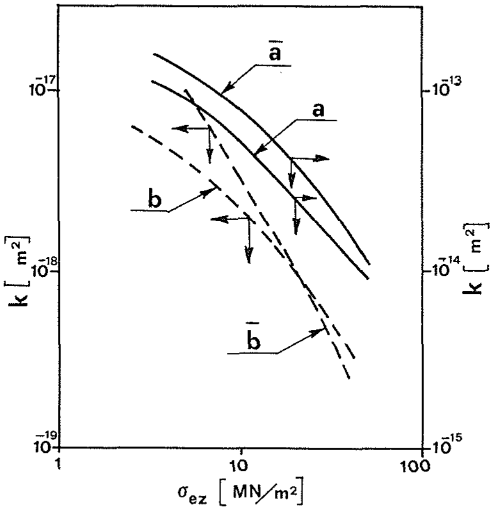

FIG. 5 k versus σ_{ez}: a - undisturbed sand, $\bar{a}$ - remoulded sand, b - undisturbed clay $\bar{b}$ - remoulded clay

4. choice of the parameters as a function of the actual stresses. Considering that compressibility and permeability depend on the loading history and the actual effective stresses, ranging during the exploitation from $\sigma_{ezi} = \sigma_z - p_i$, to $\sigma_{ezf} = \sigma_z - p_f$, it is advisable to choose parameters corrisponding to the mean stress in the soil zones subjected to consider

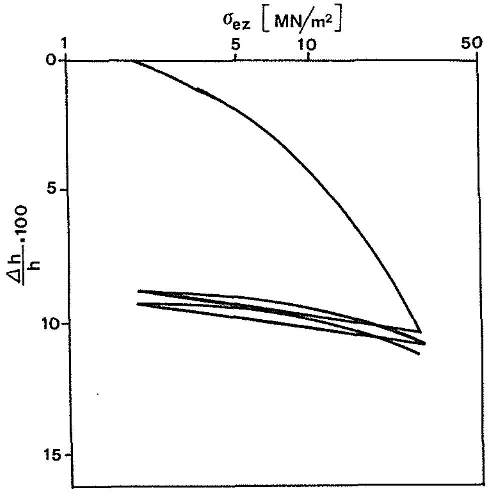

FIG. 6 Hysteresis in compressibility of a sand-core

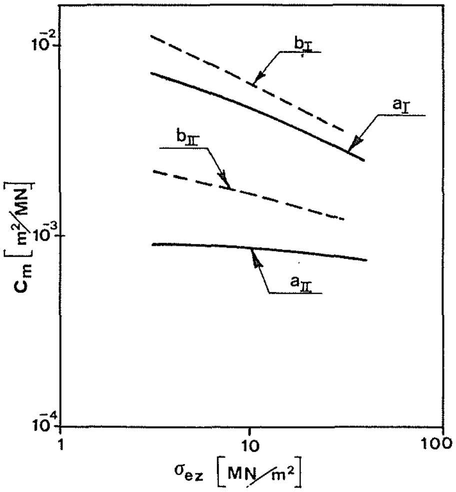

FIG. 7 C_m versus σ_{ez} for first and second loading cycles: a - sand, b - clay

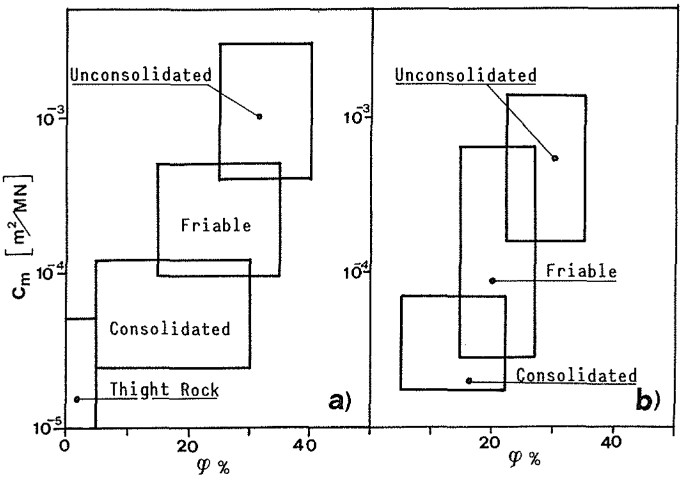

FIG. 8 Range in uniaxial compaction coefficient for consolidated friable and unconsolidated sands: a - Teeuw 1973, b - Newman 1973

able decreases in pressure, while parameters corresponding to the actual initial stress are chosen far from the exploited layers, considering that there the actual stresses vary slightly.

Approximate compressibility values in the Po Valley

The values of permeability and compressibility of loose materials depend on several parameters. If we consider them as a function of some parameters only, we would obtain very approximate values as regards compressibility, while we wouldn't be provided with any correlation regarding the permea-

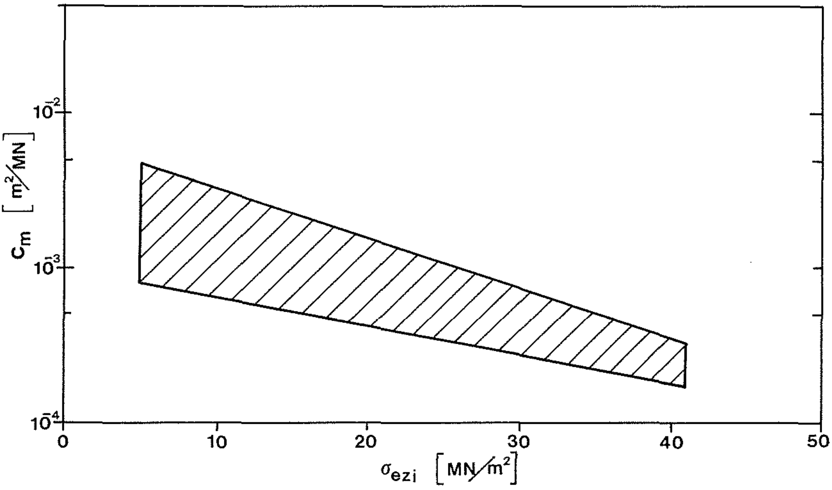

FIG. 9 Range in uniaxial compaction coefficient for unconsolidated rocks in the Po Valley

bility, which is extremely sensitive to the slightest variation of both grain size distribution and soil structure.

In order to have an idea about the possible values, we have reported in Fig. 8 some ranges comprising most values of C_m obtained by Newman (1973) and Teeuw (1973) for sandstones, slightly cemented sands and loose sands. It should be noted that the original data were modified to make them homogeneous. These values are extremely scattered and, therefore, cannot be used even for an approximate evaluation.

However, if the measures refer to a limited área with similar geologic and lithologic characteristics, a good correlation between depth and compressibility can be found.

By widening such area, a range can be determined comprising most samples examined.

In the case of the Po Valley, with a depth range from 500 m to 3,000 m (and leaving aside the values of clays in fresh water), the measures taken on about 80 samples allow us to determine a range comprising more than 80% of the values of C_m as a function of σ_{ezi} (Fig. 9). It was not possible to decidedly distinguish the domain of clays from the domain of sands. As a rough guide, it could be stated that clays are more frequent in the upper part of the belt, while sands in the lower part. On the basis on many tests, it was also possible to see that the Poisson's rations range from 0.2 to 0.3.

Conclusions

The previous paragraphs have pointed out the main causes of error when determining the compressibility and permeability of unconsolidated materials in situ.

Such difficulties suggest that the investigation should be concentrated on the soil zones which mostly influence the phenomenon of soil subsidence.

While it was impossible to find correlations which could provide the permeability of the medium as a function of few parameters (in this case even the possibility of using the laboratory tests is in doubt), for the compressibility it was possible to find approximate correlations, depending on both the lithologic characteristics and the stress system applied to the medium, valid for limited areas.

Obviously, such correlations provide only approximate values which will have to be more accurately determined by a specific investigation.

It is estimeed, however, that also these last values are not sufficient for a final forecast. Therefore, they should be more precisely defined according to the known trend of subsidence. On this connection, Holzer (1981) found that the compressibility values may vary in time when the compressibility stresses exceed the preconsolidation stresses. It is therefore advisable to go forward warily when extrapolating the results obtained.

It is also worth noting that, when subsidence is due to several causes, each acting at different depths (e.g. lowering of the phreatic surface, depressurization of aquifers, production of hydrocarbon reservoirs) it is sometimes impossible to assess the quantitative effect of each contributory cause only on the basis of the measurement of subsidence. In such cases it is necessary to evaluate the compaction of the different strata separately through in situ measurements.

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GROUND MOVEMENTS CAUSED BY MINING ACTIVITIES IN THE NETHERLANDS

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Abstract

The mining activities in the Netherlands consist of coal mining from the beginning of this century to mid seventies and in the present the production of oil and gas on- and off-shore, particularly from the major Groningen gasreservoir.

Small and deeply buried gas reservoirs are hardly capable of producing notable subsidence, in contrast to extremely large gas reservoirs at great depth, like the major Groningen gasreservoir, that may be potential candidates for ground movements.

Subsidence results from the interaction between the compacting reservoir and its visco-elastic surroundings. This displacement interaction can be calculated for a disc-shaped reservoir using the theory of poro-elasticity.

An outline is given of the subsidence due to coal mining in the past. After the termination of the coal extraction and the ceasing of the pump-activities, the mine water level has gradually risen to the surface. Surface movements above the mine water reservoir result from the interaction between the dilating reservoir and its elastic surroundings, which are mutually connected. The surface rising is treated as a typical problem in elasto-mechanics, where the mine water reservoir dilates in a half space, with a traction-free surface, due to an increase in pore-pressure up to hydrostatic level.

General

The last twenty years the domestic energy supply of the Netherlands changed dramatically. This change has been strongly determined by the discovery of the Groningen gasfield with an expected reserve of $2500 * 10^9\ m^3$. Until the end of the fifties the energy needs of the Netherlands were mainly supplied by coal of national origin. Coal mining was concentrated in the South of the Netherlands, with an average annual production of $12 * 10^6$ metric tons. The total exploited quantity of coal amounted to $600 * 10^6$ tons, and was mainly exploited from 1900 to 1975. This coal-mining caused substantial land subsidence which in several places amounted to more than 10 meters and in many cases it lead to heavy mine-damage. For an overview of the total surface subsidence due to coalmining see figure 1.

Since 1965, the Groningen gasfield is in production and the total exploited quantity of gas comes to more than $800 * 10^9\ m^3$. This quantity of gas is extracted from a reservoir with a thickness of 150 m, at a depth of 3000 m and with an extention of nearly 900 km^2. In 1975, the NAM (Nederlandse Aardolie Maatschappij) in cooperation with KSEPL (Koninklijk Shell Exploratie en Produktie Laboratorium) published the results of an extensive investigation,which shows that the surface subsidence above the Groningen gasfield amounts to more than 25 cms in the far future. This slight surface subsidence still makes it necessary to make provisions for

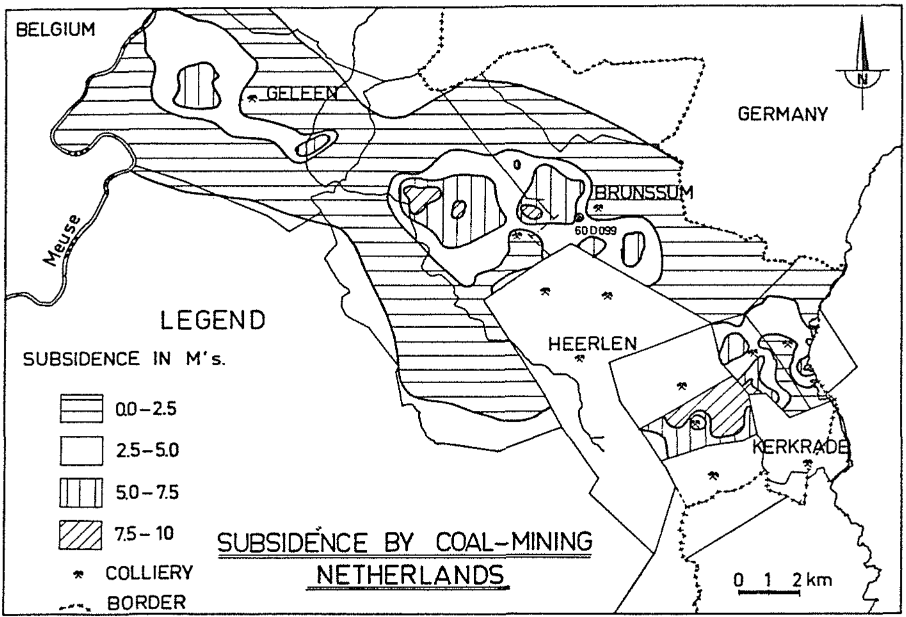


FIGURE 1

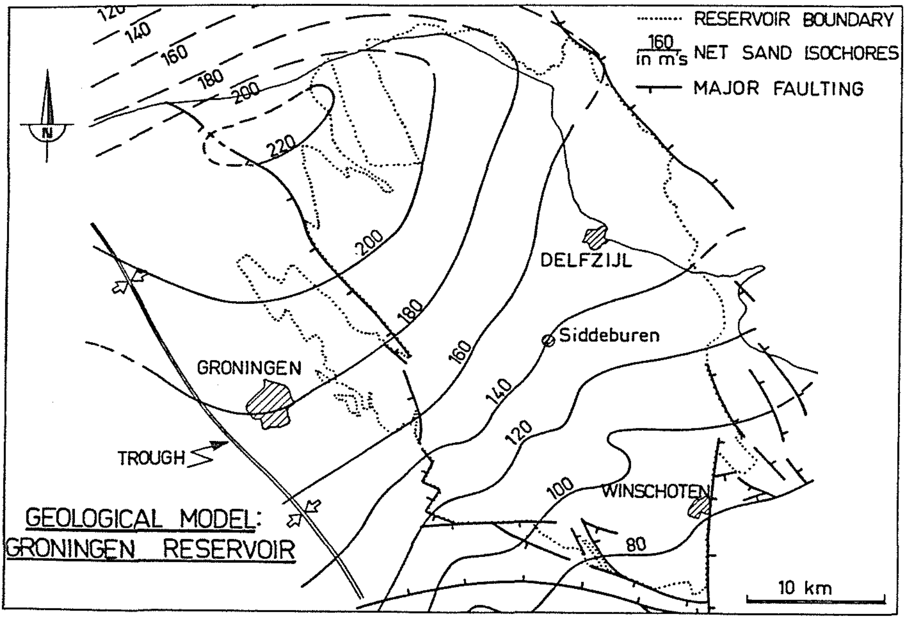


FIGURE 2

mainting the vulnerable water-management in the polder-region of the Groningen province, most of it being situated below mean sealevel. For the Groningen-reservoir see figure 2.

For the Groningen gasfield a comparison is made between the prediction of the subsidence on the basis of an elastomechanic calculating model and the results of an annual large scale levelling survey. Additionally this levelling survey, subsidence is controlled by shallow compaction recording, by means of a cable-measurement method, and by monitoring of radio-active bullets for measuring the in-situ reservoir compaction.

After the abandonment of the Limburg coalmines, the pumping of mine water has ceased and the old workings, covering an area of 160 km^2 and in depth reaching from the surface to nearly 1000 m, are gradually being flooded. The rising of mine water is mainly caused by a waterflow from greater depth which causes a surface rising by a pressure-build up in the subsurface. The pressure-build up will in the end rise to the hydrostatic level and it is anticipated that a surface movement of 20 to 30 cms will develop.

In order to make the necessary provisions in time as well above the Groningen gasreservoir as above the mine water reservoir in Limburg, a reliable prediction of the subsidence or surface rising to be anticipated is necessary. In both cases, a phenomenological prediction-model is used; where an isolated volume - the reservoir - shrinks or dilates in a half-space with a tractionfree surface due to a reduction, or build up in pore-pressure.

The displacement can be calculated for arbitrary geometrical reservoir conditions applying the theory of pore-elasticity.

Compaction or dilatation of disc-shaped reservoirs

The mechanism of reservoir compaction or dilatation can be explained as follows. The pressure of producible gas or incoming water is mainly dependent on the depth of the accumulation. The degree of compaction or dilatation depends on the strength of the formation matrix which is related to the effectiveness of the cementation of the rock-matrix. The total compaction or dilatation Δh is therefore related to the deformation properties of the rock, the reduction or increase in the gas- or fluid-pressure Δp , and the thickness h of the formation. The geometry for the determination of the displacement field around a disc-shaped reservoir in the half-space, is given in figure 3. In order to predict the compaction or dilatation behaviour the following information is required:

- a map, showing the full extent of the reservoir area and also indicating the reservoirthickness (h),
- a prediction of the reservoir pressure reduction or increase (Δp) at a given time,
- the uni-axial compaction coefficient c_m or the uni-axial dilatation coefficient d_m of the rock, i.e. the compaction or dilatation per unit stress.

Hence the total compaction or dilatation at a given time and place can be calculated from the formula:

$$\Delta h = (c_m \text{ or } d_m) * \Delta p * h$$

The surface subsidence or rising above the centre of a disc-shaped reservoir amounts to:

$$u_z\,(0,0) = \mp 2(1-\nu) * \left(1 - C/\sqrt{1+C^2}\,\right) * (c_m \text{ or } d_m) * h * \Delta p$$

where $C = c/R$ then c is the depth and R the radius of the disc-shaped reservoir and ν = 0,25 the Poisson-ratio. For a complete mathematical treatment of the problem see ref. Geertsma J. a.o..

A linearly changing stress $S_1 t$ is applied to a substance, which is elastic in hydrostatic compaction or dilatation and behaves as a Kelvin substance in shear.

With the application of the Correspondence principle from rheology and the theory of the Laplace transforms, the solution for the surface subsidence

Geometry for the displacement field around a disc-shaped reservoir in the half-space

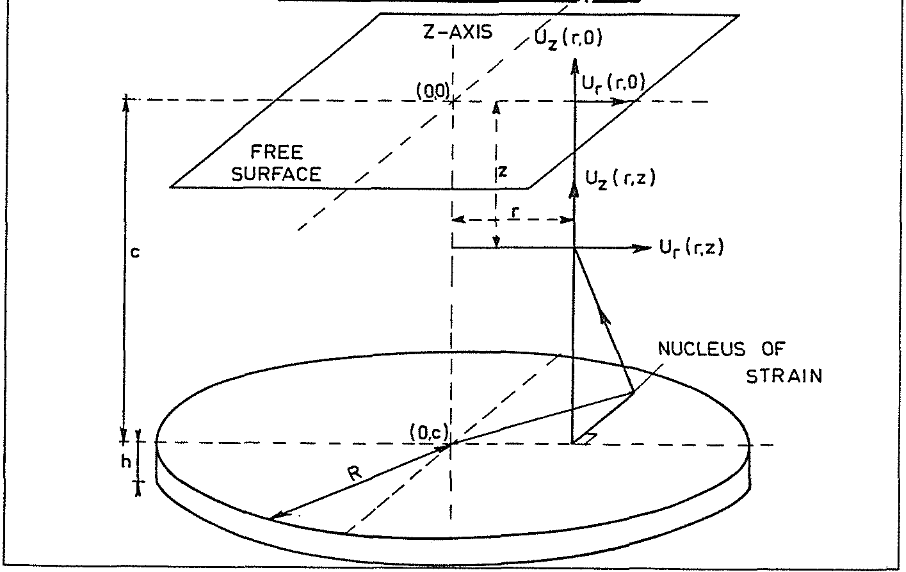


FIGURE 3

Relationship between the elastic and visco-elastic deformation above a disc-shaped reservoir

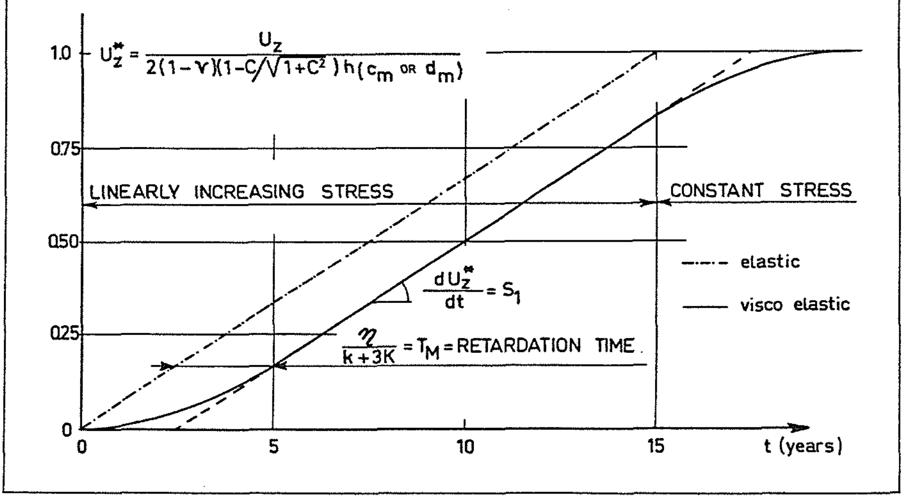


FIGURE 4

or rising in the visco-elastic half space can easily be written down from the solution in the elastic half space:

$$u_z \underset{\text{visco-elastic}}{(0,0)} = u_{z\,\text{elastic}} \left\{ t - T_M \left(1 - e^{-t/T_M} \right) \right\}$$

and T_M is the retardation time in years, see figure 4, ref. Jaeger J.C. and Cook N.G.W.

For practical prediction of subsidence or surface rising an accurate determination of the uni-axial compaction or dilatation coefficient is needed.

This can be done by analysing a series of levelling results in connection with the pressure reduction for the Groningen area or the mine water rising in the former coal-district.

The geometry of the gas- and mine water-reservoir

The subsidence or rising of a bench-mark on ground surface is strongly dependent on the shape, the extent and the depth of the reservoir, expressed in the geometry-factor $\left(1 - C / \sqrt{1 + C^2} \right)$. The subsidence or the rising is maximum with an infinite extended reservoir C = 0.

The vertical ground movement of a disc-shaped reservoir with a radius equal to the depth of the reservoir C = 1 amounts to 28% of the movement by a reservoir with an infinite radius. On the basis of the foregoing a rotation symmetrical integration-net has been drawn-up in which the percentages of influence change from 28% by C = 1 to 4% by C = 0,2, see figure 5. For practical use this integration-net is subdivided in octants, making it possible to determine, in an accurate way, the geometry-factor of a reservoir with an arbitrary shape.

In the case of the rising mine water, reservoirs are situated on top of each other, depending on the number of extracted coal-seams. It is justified to assume that the rising surface, as a consequence of the inflowing mine water, is in relation with the subsidence caused by coal-mining in the past.

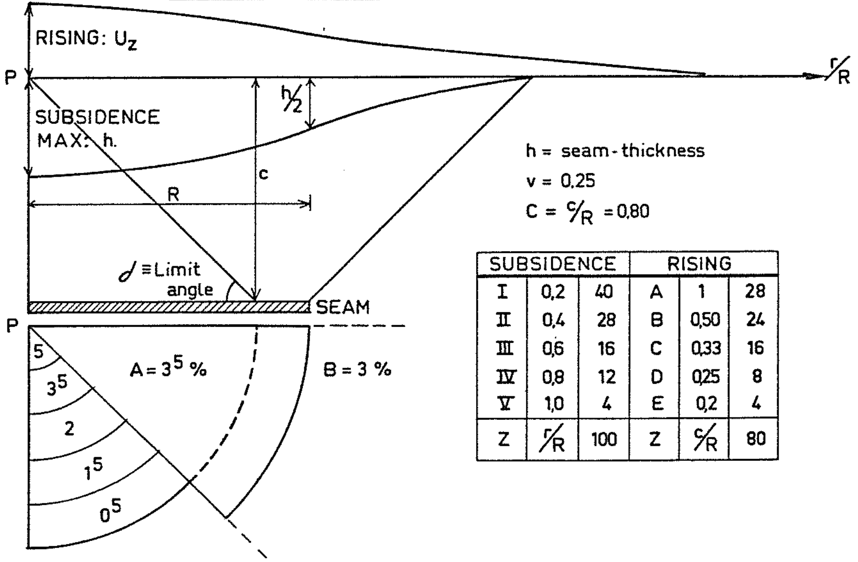
This subsidence is maximal by extracting a circular area with a radius equal to the seam-depth, the critical area.

This critical area is also subdivided in 5 zones and 8 sectors, by which the subsidence from the extraction of a coal-seam of irregular shape can be determined in a reasonable and accurate way, see also figure 5.

In predicting the surface rising in relation with the flooding of the coal workings, the assumption is made that the thickness of the delating reservoir corresponds with the original coal-seam thickness. This assumption of course influences the determination of the uni-axial dilatation coefficient, but has in the end no influence on the prediction of the surface rising.

Figure 5 shows, in the form of a nomogram, the relation between the subsidence in the past and the anticipated surface rising in the near future. For example: the extraction of a coal-seam, with a geometry-factor coal of 30%, and a geometry-factor water of 57%, and a depth of 250 m, will cause a surface rising of 11% of the subsidence through coal extraction in the past. It is to be expected that the surface in several regions of the former coal-district in Limburg will rise to more than 25 cms depending on the subsidence in the past due to coal extracting.

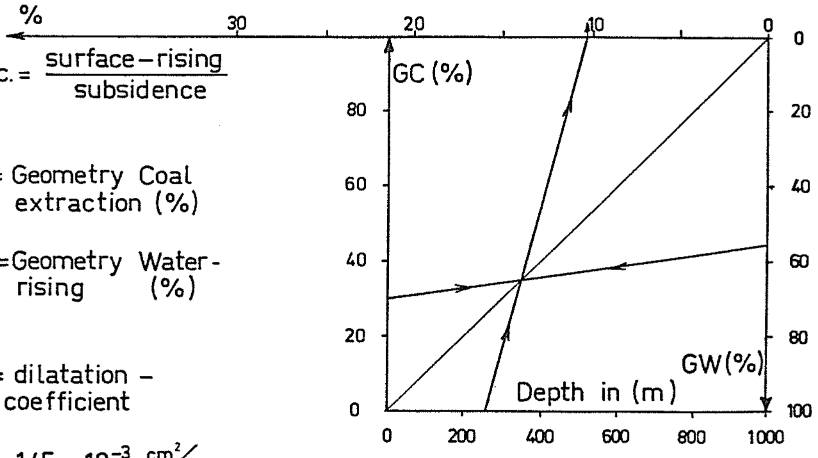
SUBSIDENCE VERSUS SURFACE-RISING



h = seam-thickness
v = 0,25
C = c/R = 0,80

SUBSIDENCE			RISING		
I	0,2	40	A	1	28
II	0,4	28	B	0,50	24
III	0,6	16	C	0,33	16
IV	0,8	12	D	0,25	8
V	1,0	4	E	0,2	4
Z	r/R	100	Z	c/R	80

SUBSIDENCE VERSUS SURFACE-RISING



Perc. = $\frac{\text{surface-rising}}{\text{subsidence}}$

GC = Geometry Coal extraction (%)

GW = Geometry Water-rising (%)

d_m = dilatation - coefficient

$d_m = 1{,}45 * 10^{-3}\ \mathrm{cm^2/kgf}$

FIGURE 5

The uni-axial compaction of the Groningen gasreservoir

The phenomenon of reservoir compaction is controlled by the volumetric changes in a porous rock as a result of the change of in-situ stresses that accompany the production of reservoir fluids or gasses. Various measuring techniques have been developed for determining rock deformation under external pressure. The tri-axial compaction test with zero-lateral strain is used for the determination of the uni-axial compaction coefficient c_m . Taking into account the different compaction behaviour of the formation-samples, an average value $c_m = 1.45 \ast 10^{-5}$ cm^2/kgf was obtained. The individual measurements were within a range of an $0.5 - 4.5 \ast 10^{-5}$ cm^2 /kgf.

As a consequence of the inhomogeneous composition of the gasreservoir and the anisotropy of the Rotliegendes sandstone, the determination of the c_m value in the laboratory shows a wide spread. Therefore, it is to be preferred to determine the uni-axial compaction coefficient in an indirect way, by the comparing of the predicted and measured surface subsidence. For this comparison and for the calculating results, see figure 6. Two geo-mechanical hypotheses are tested:

- a linear-elastic model $c_m = 0.35 \ast 10^{-5}$ cm^2 /kgf. . and
- a visco-elastic model, with the same compaction coefficient and a retardation time T_M of 1 year.

The best adjustment is reached by the assumption that the surface reacts to the visco-elastic model, with the c_m and T_M values already mentioned.

This c_m determination on the basis of field-observations differs significantly from that of laboratory-investigations and has resulted in a reduction of the predicted surface subsidence.

A dilating mine water reservoir

The uni-axial dilatation coefficient for the surface rising, in relation with the rising mine water in the former coal-district of the Netherlands, is determined by field-measurements with respect to surface rising and direct gauging of rising mine water in former shafts. For prediction of long term surface rising 3 geo-mechanical models are tested:

- a linear-elastic model, assuming the mine water rising in the Carboniferous-strata goes by a constant velocity and is not influenced by the coal extraction in the past. The best adjustment between recent observations and the prediction is obtained with an uni-axial dilatation coefficient $d_m = 0.5 \ast 10^{-5}$ cm^2/kgf . The ultimate rising according to the linear-elastic model amounts to 16 cms for the benchmark 60 D 099, see figure 7.
- a visco-elastic model with a retardation-time of 2 years. The uni-axial dilatation coefficient in this model is equal to $0.75 \ast 10^{-5}$ cm^2/kgf. Testing the visco-elastic model gives a better smooth adjustment between the observed and predicted surface rising than a linear elastic model, see figure 7.

On the basis of the tested visco-elastic model, the maximum rising in 60 D 099 will amount to 24 cms about 1990, one year after the mine water has risen to the surface.

The coal extraction in the past is related to the recent rising of mine water. Due to the long-wall caving system and the movement on a large scale that is connected with it, an additional porosity and permeability has been created. The pressure build up caused by rising mine water shapes a dilating reservoir in the old workings. The rising of the surface is, among other things, determined by the depth and the thickness of the expanding reservoir. On the basis of in-situ observations the assumption is justified that the

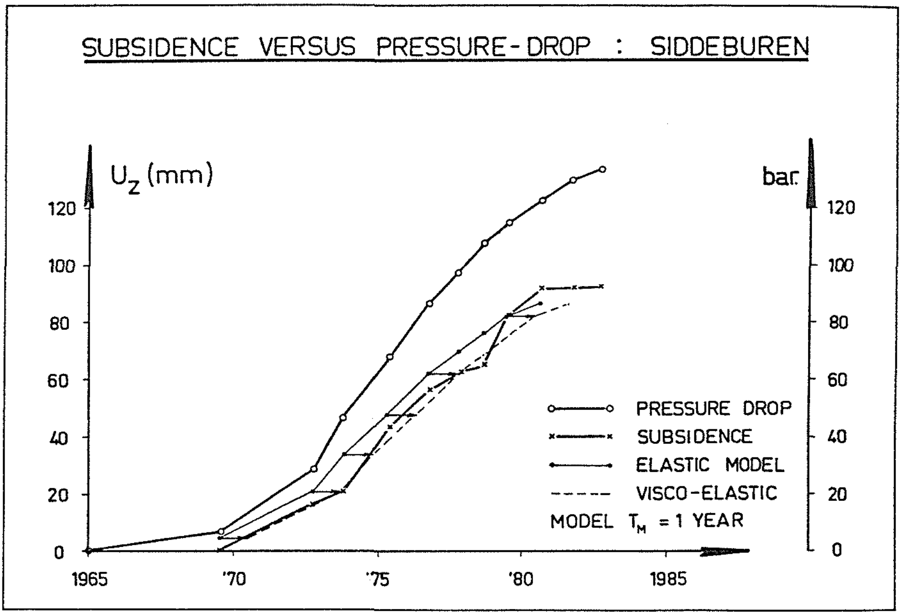


FIGURE 6

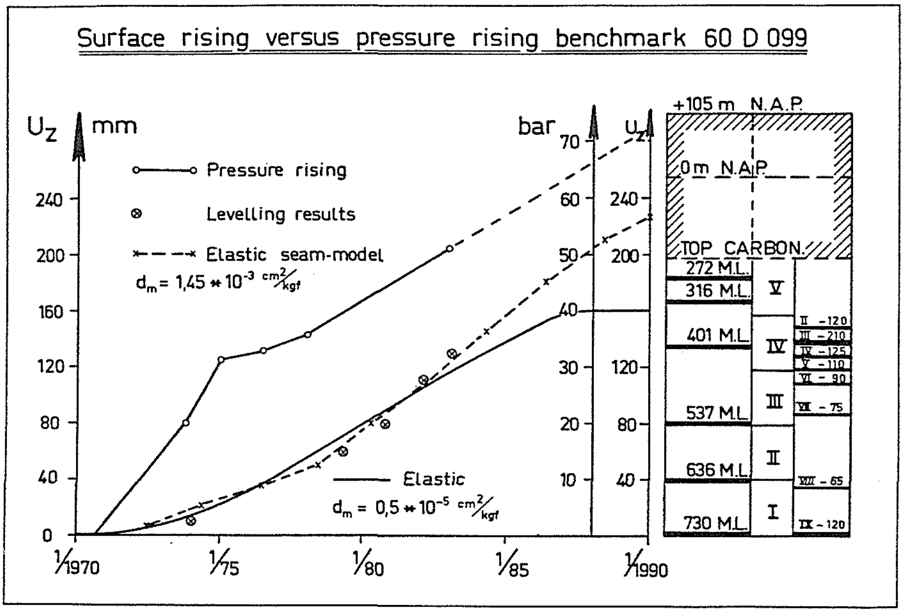


FIGURE 7

thickness of the reservoir is proportional to the seam-thickness of the coal excavation in the past. In determining the uni-axial dilatation coefficient the fore going assumption must be taken into account, and therefore a direct comparison with the coefficients in the preceding geomechanical models is impossible.

- In the model on the basis of the excavated coal-seams only a linear-elastic hypothesis with respect to the ground movements is tested, by which the pressure build-up in the reservoirs is derived from the gauging in the former mine-shafts. The best fit between observations and prediction results in an uni-axial dilatation coefficient of $1.45 \ast 10^{-3}$ cm^2/kgf.
  The maximum rising of the bench-mark 60 D 099 will amount to 23 cms and is nearly the same as in the linear elastic model with a constant rising of the mine water and a dilatation coefficient of $0.75 \ast 10^{-5}$ cm^2/ kgf and a reservoir thickness including the total Carboniferous-strata.

Conclusions

Summarising,it can be said that for the prediction of the subsidence, resulting from gas production as well as the rising of the surface by a dilating mine water reservoir, the use of a linear-elastic deformation model has to be preferred. The retardation times are very short in comparison with the total duration of the geo-mechanical processes.

In the case of the Groningen gas-reservoir,the most probable value of the uni-axial compaction coefficient is equal $0.35 \ast 10^{-5}$ cm^2/kgf. On the basis of this compaction coefficient, the maximum subsidence above the centre of the reservoir amounts to 25 cms. This subsidence will be reached around the year 2025, under normal production progress.

The most reliable prediction for the rising above the Limburg mine water reservoir is obtained by using the so-called "coal-seams model" with a linear-elastic deformation and an uni-axial dilatation coefficient of $1.45 \ast 10^{-3}$ cm /kgf. The maximum surface rising is dependent on the coal excavations in the past and in several places will amount to more than 25 cms. It is obvious that this surface rising, in relation with the gradual and smooth nature of the movement, does not cause any new mine damage.

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SUBSIDENCE RESEARCH IN INDIA.

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Abstract

Due to lack of know-how of subsidence behaviour of Indian Coal Measures seams underneath surface properties have mostly remained unexploited. The research by the Central Mining Research Station, Dhanbad has made it possible to partially develop subsidence indices and also to extract more than 6-million tonne of coal underneath surface properties. Subsidence profiles have been continuous, discontinuous as well as assymmetrical with different shapes. Approaches have been suggested for anticipation of assymmetrical profiles of different shapes for sub-critical and critical widths. The subsidence movements were greatly influenced by the percentage sandstone in superincumbent strata. The non-effective width increased with the percentage of sandstone. The maximum subsidence over hydraulically sand stowed workings was less than 5 percent of extraction thickness.

Hydro-pneumatic stowing system for bling back-filling of unapproachable old workings is being developed. The first field trial has shown encouraging results.

Programmes for extensive investigations have been drawn.

Introduction

Although the importance of subsidence research in India was realised as long back as the beginning of the century, systematic investigations were started in the sixth decade by the Central Mining Research Station (CMRS), Dhanbad. The research gained impetus after nationalisation of coal mines in 1971 and 1973. The investigations have been completed over 31 workings and are in progress over 45. Even today there is a lot to be done and programmes have been drawn for extensive investigations for next 15 years.

The coal measures in India mainly belong to Gondwana and Tertiary periods and the total indicated reserves up to a depth of about 600 m are about 83-billion tonne. Recent explorations have indicated some more reserves. The coalfields having multiple number of seams are spread over a large area in the eastern and central parts. Mining activities underground have been mainly confined in upper seams at comparatively shallow depths. There are only a few mines deeper than 500 m. The predominent method of mining has so far been bord and pillar. The contribution of longwall system was just over one percent of the total in 1982-83, which was approximately 133-million tonne. The share of underground mining being about 83-million.

In almost all the coalfields problems are being faced in exploitation of seams due to presence of surface properties, e.g., roads, railways, rivers, ponds, buildings, etc. The subsidence research conducted so far has been helpful in extracting more than 6-million tonne of coal underneath surface properties. A main railway line at Sudamdih in Jharia coalfield has been made to subside gradually by a maximum of 448 mm, while three seams dipping at 1 in 2 were extracted with hydraulic sand stowing.

The past mining practices have left a legacy of unapproachable, unknown, waterlogged and dry old workings posing danger to surface properties due to collapse of pillar remnants. Many sudden collapses have taken place in such workings resulting in extensive damages to surface properties due to subsidence movements. Hydro-pneumatic system for blind backfilling is being developed for stabilising such workings.

Indian Coal Measures

The coal measures in Indian coalfields consist of mainly sandstones, shales, and sub-soils. In some areas the thickness of soils/clays is more than 250 m. The sandstones vary from coarse to very fine grained and the thickness of individual beds up to 80 m and more. The shales, finely bedded, interlace sandstone and coal beds and their percentage is up to maximum of about 40.

The maximum thickness of a coal seam is about 150 m (Jhingurda seam in Singrauli). The general thickness being 3 m to 9 m. Seams up to a thickness of 1.2 m are in general being considered unworkable. The thick seams have been extensively developed in one or more sections on bord and pillar pattern.

Coal Winning

The most predominent method of underground winning of seams is bord and pillar. The longwall system has so far found very limited application. Mechanisation of mines has been on a very modest scale.

The mining activities underground are confined to shallow depths in general. Extraction of thick seams developed in two or more sections has been of concern to most of the mining engineers in the country.

Subsidence Profiles

Visually both continuous and discontinuous subsidence profiles have been observed in Indian coalfields (Fig.1). Stepped subsidences have mainly taken place over thick seams extracted by bord and pillar system under shallow covers having sandstones predominently. The subsidence profiles observed were not symmetrical and their shape also varied. The norms and methods used in other countries were in general not suitable for anticipation of subsidence movements. On the basis of experience in field investigations the following equations have been suggested for anticipation of

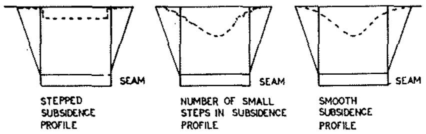

Fig. 1 - Nature of subsidence profiles.

subsidence movement profiles for sub-critical and critical widths, which can be used to obtain assymmetrical subsidence profiles of different shapes and they satisfy boundary conditions also.

For sub-critical widths (Fig. 2), the boundary conditions are

$$x = 0,\ s = S,\ g = 0 \text{ and } 1/\rho = \text{some value} \qquad (i)$$
$$x = b_1 \text{ or } b_2,\ s = 0,\ g = 0 \text{ and } 1/\rho = 0 \qquad (ii)$$

and the equations suggested are

$$s = S\, e^{-n \frac{x^2}{b^2 - x^2}} \qquad \ldots (1)$$

$$g = -s \frac{2\, b^2\, x\, n}{(b^2 - x^2)^2} \qquad \ldots (2)$$

$$1/\rho = g \frac{2x^2b^2 + b^4 - 3x^4 - 2nx^2b^2}{x\ (b^2 - x^2)^2} \qquad \ldots (3)$$

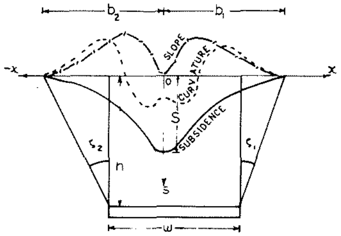

Fig. 2 - Subsidence, slope and curvature profiles for sub-critical widths.

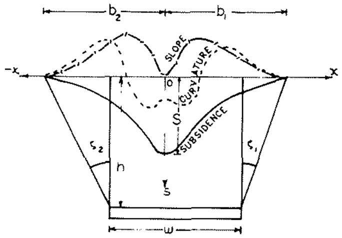

Fig. 3 - Subsidence, slope and curvature profiles for critical widths.

For critical widths (Fig. 3), the boundary conditions are

$$x = 0,\ s = S,\ g = 0 \text{ and } 1/\rho = 0 \qquad (i)$$
$$x = b_1 \text{ or } b_2,\ s = 0,\ g = 0 \text{ and } 1/\rho = 0 \qquad (ii)$$

and the equations suggested are

$$s = S\ e^{-n \frac{x^4}{b^4 - x^4}} \qquad (4)$$

$$g = -s \frac{4\ b^4\ x^3\ n}{(b^4 - x^4)^2} \qquad (5)$$

$$\frac{1}{\rho} = g \frac{x}{(b^4 - x^4)^2} (3b^8 - 5x^8 + 2b^4x^4 - 4nx^2b^2) \quad \ldots (6)$$

In the above equations s, g and $\frac{1}{\rho}$ are the subsidence, slope and curvature at any point at a distance x from the origin, S is the maximum subsidence, b is length of each flank of subsidence profile, and n is a factor controlling the shape of the profiles. Assymmetrical profiles can be plotted by using two different values of b for the two flanks.

The values of n for sub-critical cases examined so far has been between 1.0 and 15, and in case of a critical width area having 24-30 m quarry overburden debris on surface it was found to be 30.

Non-effective Width

The subsidence investigations in Indian coalfields have been confined to areas having single seam workings, in which a phenomenon of delayed subsidence was observed. The minimum width causing start of subsidence has been termed as 'non-effective width' and is expressed in terms of depth. It was found to be 0.3 to 1.0 times the depth in different coalfields and was influenced by the percentage of sandstone in the overlying strata as seen from Fig. 4. The percentage of sandstone tends to increase the non-effective width.

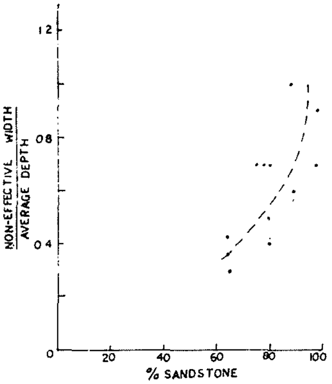

Fig. 4 - Relationship between non-effective width and percentage of sandstone.

Angle of Draw

It was earlier believed that the angle of draw in Indian coal measures is negative, i.e. the area of subsidence on surface is less than that extracted underground. This view was formed mainly due to lack of information. The investigations conducted have shown that the area influenced on the surface was always more than that extracted underground, thereby the angle of draw is positive.

The maximum angle of draw in case of flat seams has been between 20° and 25°. It was generally more on the starting side than on the finishing side, probably due to the fact that the first break of superincumbent strata takes place in the condition of being supported on all the four sides and subsequent breaks take place in the state of having support on three sides. This phenomenon is also responsible for assymmetrical subsidence profiles.

The percentage and the thickness of sub-soils and loose rocks in the superincumbent strata caused increase in the angle of draw. The maximum being about 40° in an area having about 150 m of clays.

In case of dipping seams the angle of draw was more on the dip side than on the rise side, and the subsidence movements were in confirmation with normal theorey.

Maximum Subsidence

The maximum subsidence observed over hydraulically sand stowed longwall and bord and pillar workings has been about 5 percent of extraction thickness. Fig. 5 shows the maximum subsidence over stowed workings plotted against their width-depth ratio and Fig. 6 shows a similar plot for caved workings. The figures do not indicate any relationship between these two parameters. In Fig. 7 percentage of sandstone has been incorporated with these parameters for caved workings, which shows that the percentage of sandstone also influences the magnitude of subsidence.

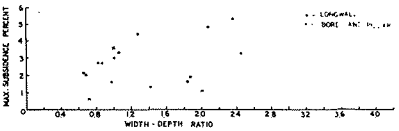

Fig. 5 - Maximum subsidence vs width-depth ratio over stowed faces.

Relationship between Subsidence Parameters

In the following general relationships between maximum subsidence (S), slope (G), and strain (E) and average depth (h) the values of the constants were computed from the field observations, which are plotted against width-depth ratio in Fig. 8. It can be seen that k1 tends to decrease with the increase in the width-depth ratio, whereas k_2 and k_3 tend to increase.

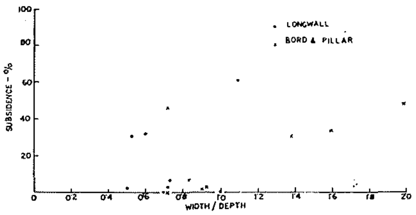

Fig. 6 - Maximum subsidence vs width-depth ratio over caved faces.

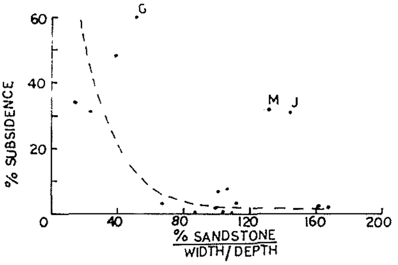

Fig. 7 - Relationship between maximum subsidence, percentage sandstone and width-depth ratio.

$$G = k_1 \; S/h \qquad \ldots (7)$$

$$E_{(-)} = k_2 \, G \qquad \ldots (8)$$

$$E_{(+)} = k_3 \, G \qquad \ldots (9)$$

Safe Limits of Subsidence Movements

The experience in field investigations led to provisionally defining the safe limits of subsidence movements for railway lines, buildings and water bodies.

Railway Lines - For regularly jointed rail construction the maximum permissible strain is 3 mm/m. The limiting operating gradient is 1 in 100 or 10 mm/m.

Buildings - The maximum permissible compression or expansion in the buildings is 60 mm, which is expected to cause slight damage which can be easily repaired.

Water bodies - The maximum permissible tensile strain in the beds of water bodies is 3 mm/m. The limit has been defined considering the nature of superincumbent strata in Indian coalfields and the nature of beds of water bodies, which are sandy in general.

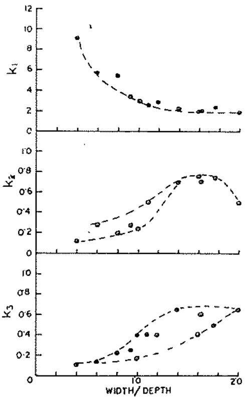

Fig. 8 - Relationship between k_1, k_2 and k_3 and width-depth ratio.

Case Studies

The know-how of subsidence behaviour of Indian coal measures developed after field investigations, described earlier, has been used in studying 48 problems involving extraction of coal seams underneath surface properties and protection there-of. Six problems are under study. As a result of the studies it has been possible to help the industry to extract about 6-million tonne of coal underneath surface properties. A few important examples are given hereunder.

At Sudamdih shaft and incline mines of Bharat Coking Coal Limited (BCCL) in Jharia coalfield three seams namely 7.5 m thick XI/XII seam at depth ranging between 35 m and 400 m, 3-6 m thickness in IX/X seam at depth ranging between 200 m and 300 m, and 3-4.5 m thick VIIIA seam at depth ranging between 200 m and 400 m, dipping at 1 in 2 have been extracted underneath and in the vicinity of a main railway line, Damodar river, and other surface properties by longwall ascending slicing with hydraulic sand stowing. The total quantity extracted so far is about 2-million tonne. The railway line has been made to subside by a maximum of 448 mm in a period of about 12 years without any disturbance in its normal operation. The maximum strain suffered by the line was 2.9 mm/m and its steepest long gradient after subsidence was 1 in 103. Another 4-5 million tonne of coal is to be extracted in future.

At Surakachhar colliery of Western Coalfields Limited successful extraction has been done underneath water bodies and their highest flood level and an assisted railway siding in 1.8-3.0 m thick G-III seam at depth ranging from 50 m to 140 m by longwall system with hydraulic sand stowing. The maximum subsidence observed over the stowed workings was less than 5 percent of extraction thickness.

Ten blocks in 1.7-1.8 m thick XVII-Top seam have been extracted successfully by longwall caving system at depth ranging from 220 m to 440 m mainly underneath built-up areas at Moonidih Project of BCCL. As a result of the extraction a number of buildings were slightly damaged, which were repaired at a nominal cost of Re 1/- ($ 0.1) per tonne of coal extracted. The non-effective width was found to be o,4-0.5 times the depth. The maximum subsidence observed was about 30 percent of extraction thickness for a width-depth ratio of 0.6. For the protection of surface properties in some areas the width of extraction underground was restricted to 0.4-0.5 times the depth and 0.1-0.2 times the depth wide barriers were left between the blocks.

Successful extraction underneath and in the vicinity of surface properties has also been done at Loyabad, Pootki, Dhemomain, Ningah, Bhelatand-Sijua and other collieries.

Hydro-pneumatic Stowing

Laboratory investigations on a scale model led to development of hydro-pneumatic stowing system of blind backfilling of unapproachable and unknown underground workings. Schematic diagram of the system is shown in Fig. 9. The force of compressed air being released at about 300 mm below the end of the bore hole is used to keep on breaking the formation of cone below the bore hole and also to transport solids in the voids. Due to buoyancy of air the movement of solids was more towards rise side.

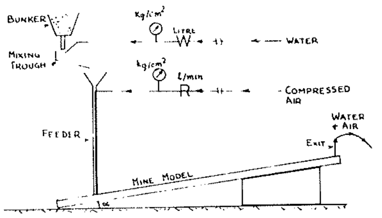

Fig. 9 - Schematic diagram of hydro-pneumatic stowing system.

The first field trial of the system for stabilisation of old workings in XIV seam at a depth of about 45 m underneath a jore bed at Jogta Fire Project of BCCL has shown encouraging results. More than 2,000 cubic metre of solids have been stowed from the first bore hole without being choked. Soon field experiments with the system would be made at a few more places.

Conclusions

Although some know-how has been developed in respect of subsidence behaviour of coal measures in Indian coalfields more developments are necessary to fulfil the existing gaps. Programmes have been drawn for extensive research for the next 15 years for this purpose.

Acknowledgements

The authors are thankful to the Director, CMRS, Dhanbad for according permission to present this paper.

The views expressed in the paper are those of the authors and not necessarily of the CMRS.

SURFACE SUBSIDENCE DUE TO WATER SEEPAGE AND THE PRESENCE OF OLD MINE WORKINGS IN SOUTHERN ALBERTA, CANADA.

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Abstract

Surface subsidence, caused by underground coal extraction and subsequent flooding of the mine by ground water, has occurred in the vicinity of the City of Lethbridge in southern Alberta, Canada. The exploited coal bed of Galt Mine No. 6 is overlain by approximately 104 m of generally incompetent layers of silt, silty clay, gravel, mud stone, shale, and silt stone. Records of mining activity in the area are incomplete, complicating the evaluation of the overall stability and safety of the site. Analysis of available subsidence data, existing subsurface conditions, and performance of developed sites suggests that if any surface subsidence occurs in the future in the study area it is likely to be negligible.

Introduction

The locations of many population centres throughout the world have been influenced by the proximity of a valuable natural resource and the availability of efficient, economic transportation. As these centres expand in size, there is often pressure to utilize the areas once exploited for natural resources for residential and industrial uses. The study site, within the City of Lethbridge, is just such an area. The city has the main line of the trans-Canada railroad passing through it and overlies coal deposits.

Large-scale coal mining in this area started in 1882. Mining under the study site, in Galt Mine No. 6, was carried out from 1908 to 1935 using the room and pillar method. The approximately 1.5 m thick coal seam is overlain by about 104 m of sediments. Mining records are incomplete, due to the remoteness of this mine from population areas during the active mining period. As the use of this land changes from farming to residential, the magnitude of potential human and economic losses, due to continuing surface subsidence, will increase dramatically. This study was carried out to estimate if surface subsidence is likely to occur which makes the site unsuitable for residential use.

Subsurface Profile

Mining records and previous field investigations indicate that subsurface soils extend to a depth of 66 m - 95 m, followed by unconsolidated shale interlayered with coal seams to a depth of about 114 m (Brown and Casey, 1971; Klohn, 1981; Hardy Associates, 1980). The uppermost strata, extending to a depth of approximately 67 m, is composed of silts and silty clays of glacial fluvial origin. These poorly consolidated deposits are of low to medium plasticity, with occasional layers of high plastic clay. Varying amounts of gravel and boulders are found in this strata, as in most glacial deposits. A 7 m - 9 m thick, generally water-bearing, layer of gravel and boulders, termed "Saskatchewan gravel", underlies the overburden soils (Klohn, 1981). This is underlain by 40 m - 42 m of relatively incompetent rocks of the Bearspaw and Oldman formations. These mud stones, shales, and silt stones are interlayered with coal and bentonite clay. The only exploited coal bed is located at a depth of approximately 104 m and averages about 1.5 m thick.

This coal seam slopes 1 in 73 on an average bearing of N 6° W and is strongly cleated with major cleavage running N 58° E. It is of firm structure being stronger than the associated bedrock (Brown and Casey, 1971). A complex system of minor faults, ground deformation, and gentle undulations is exhibited by the coal beds and associated rocks. Strata immediately overlying and underlying the coal seams are uncemented and unconsolidated or poorly consolidated. No competent bedrock has been found above the coal seams (Klohn, 1981; Hardy Associates, 1980). Recent data from subsurface investigations correlate well with old mine records and, combined with geological studies, indicate the various strata are relatively uniform and flat. These sediments were not significantly affected by tectonic forces from the mountain building activity in western Alberta (Brown and Casey, 1971).

Mining Methods and Conditions

All 5 major coal mines in the Lethbridge area used the room and pillar mining method. Typically, the rooms, which measured 6 m - 9 m wide and about 95 m long, were separated by sacrificial pillars 2.5 m - 2.8 m thick. Impurities in the coal, such as clay and shale were left in the rooms. Heavy timbering was required to protect the entryways to the rooms and the haulage tracks in the rooms. In many cases, the timbers were removed while withdrawing from the rooms to allow roof collapse and re-use of the timber (Brown and Casey, 1971; Livingstone, 1981).

Mine records indicate that some pillars were robbed while retreating from the mine, although the extent of this practice is unclear (Brown and Casey, 1971; Klohn, 1981; Livingstone, 1981). Survey maps of the mine were prepared only every 6 months, as a result only the pillars which were safely approachable, visible, and indicated by the mine boss were shown on the mine map.

Records of floor and roof conditions in Galt Mine No. 6 were not kept. Reports from nearby Galt Mine No. 8 indicate that floor heave was a common occurrence (Brown and Casey, 1971; Livingstone, 1981). Over a two-day period, floor heave as large as 25 cm to 30 cm was recorded (Brown and Casey, 1971). As timbering was withdrawn or cut out from completed rooms caving on a large scale took place (Livingstone, 1981). Available records indicate that there were differences in the floor and roof conditions of various mines in the area even though all of the mines exploited the same coal seam. Discussions with "old timers" indicate that the conditions of Galt Mine No. 6 were no different than the conditions in Galt Mine No. 8.

Water seepage into Galt Mine No. 6 was occurring prior to abandonment (Livingstone, 1981). Apparently this water seeped through the boundary pillar separating Galt Mine No. 6 from Galt Mine No. 3, which required daily pumping while coal was being extracted. Galt Mine No. 3, which may have extended beneath the Old Man River, flooded quickly after it was closed in 1924 (Livingstone, 1981).

Records and interviews indicate that cracks and sinkholes developed around the periphery of Galt Mine No. 6 during and after active mining (Livingstone, 1981). The vast majority of sinkholes and other surface subsidence features which have occurred are close to the river or valley walls which have coal outcrops. Recent borehole data from the area suggests the presence of cavities or bulked rock up to 20 m above the mine level (Hardy Associates, 1980). Survey data from a monument located over Galt Mine No. 6 indicates a drop of 11 mm from 1968 to 1978 (EBA Consultants, 1983). Other monuments in the area, but not over mine workings, showed changes in elevation of +9 mm to -2 mm. All of these figures are considered to be within the order of accuracy of the equipment.

Surface Subsidence

Surface subsidence caused by the room and pillar mining method is influenced by the rock properties above and below the mined seam, ground water hydrology, and the size and properties of pillars left for roof support (Bruhn et al, 1978; Oravecz, 1977; Piggot et al, 1977). In addition to other factors, the depth of the mine, thickness of the mined seam, size of underground opening, mining method, seam inclination, and time also play a role in surface subsidence. While several studies indicate that ground subsidence due to the longwall mining method can be accurately predicted, it has been found that the room and pillar method produces subsidence which is difficult to predict or analyze mathematically (Bruhn et al, 1978; Collins, 1977; Dahl, 1973; Orchard, 1954; Oravecz, 1977; Voigt et al, 1970).

The roof structures in the Lethbridge area are generally formed of weak shales, mud stones, and silt stones, which are not likely to be able to support free spans of 7.5 m - 9 m. As a result, the roofs yielded under the overburdened loads almost immediately. Similarly, the extra load on the pillars caused the weak underlying sediments to squeeze, resulting in floor heave. Failure of the roof and floor materials is likely to show up quickly as surface subsidence. Generally, unless the overlying rocks are sound, massive, and not extensively jointed, surface subsidence shows up almost immediately (Orchard, 1954). Softening of the roof and floor materials by the water which flooded the mine may have helped this process.

Underground voids are likely to exist near timbers and pillars which were not removed as even weak rocks may span short distances. Water seepage into these voids, along failure surfaces and through tension cracks, may erode and wash down overlying sediments. A void close to the surface may result which is too large for the overlying soils to span. When the overburden collapses into this void, a sinkhole is formed (Bruhn, 1978; Piggot et al, 1977). Sinkholes occur particularly in areas where the cavities are located within 30 m of the surface (Bruhn, 1978; Hunt, 1979). This view is supported by the observation that almost all of the sinkholes associated with Galt Mine No. 6 occurred in the river valley where the overburden is relatively thin.

Cavities remaining at mine level may also migrate upwards as roof materials progressively collapse into it. These broken rocks bulk in the void, thus filling a larger volume than the rock in its natural state (Herbert et al, 1927). Bulked rock usually chokes the void, unless the material is being washed away by water deeper into the mine. Typical bulking factors for shales and soft beds are 40% - 80% (Blyth et al, 1974). It has been estimated that bulking limits the distance a void can migrate to 8-10 times the height of the mined seam (Blyth et al, 1974; Taylor, 1975). Borehole data over Galt Mine No. 6 indicates voids have migrated approximately 13 times the mined seam height. This suggests that water may be washing some of the bulked material further down into lower voids.

A study of surface subsidence over Galt Mine No. 8 was carried out by Brown and Casey while the mine was in operation from 1951 to 1956 (Brown and Casey, 1971). As noted previously, the conditions in this mine were very similar to that of the mine under the study site. Although the room and pillar mining method was also used, no attempt was made to rob the pillars in Galt Mine No. 8. Instantaneous settlements recorded comprised 80% - 90% of the maximum observed. The time factor for total settlement was 2-3 years. Maximum surface subsidence of 29% occurred over the centre of a room. Maximum subsidence of 30% to 40% of the cavity height have been reported for the medium extraction rates typical of room and pillar mining methods, particularly in cases where the mine is deeper than 50 m (Hunt, 1979). This suggests that further subsidence after the study period is unlikely to be more than 30% of that which has already occurred or about 15 cm. The practice of robbing the pillars in Galt Mine No. 6 may have accelerated the rate of subsidence as the overlying strata would have to span an even larger distance.

Two single-storey structures (farmhouses) are present over the old mine workings, of which one is 20-30 years old and the other over 60 years old. No sign of adverse effects from surface subsidence is apparent in either of these structures.

Conclusions

All major surface subsidence at the study site due to the extraction of coal from Galt Mine No. 6 is most likely to be complete. This major subsidence probably occurred within a maximum of 5 years after the mine closure, or almost 45 years ago. The presence of subsurface voids indicates that future sinkhole development cannot be completely ruled out. However, none of these voids were found within 80 m of the surface, so it is very likely that continued bulking of material collapsing into the voids will eventually choke them completely.

Analysis of available subsidence data, existing subsurface conditions, and performance of existing structures, suggests that the study area may be developed with minimal risk. One to three-storey structures should experience little or no damage due to settlements which may still occur. The minimal risk may be further reduced by using telescopic joints and flexible pipes for utilities which should be buried separately.

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SUBSIDENCE OF ABANDONED LIMESTONE MINES IN THE WEST MIDLANDS OF ENGLAND

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Abstract

Abandoned limestone mines in the West Midlands of England present potential problems of subsidence under urban areas. A two year study has been made.

i) to discover the extent of information that remains,
ii) to investigate the extent of the mines and physical characteristics of the mines and the rocks surrounding them
iii) to establish the degree of risk, if any, of ground movement occasioned by the collapse of the mines.
iv) to consider and recommend remedial and other works for dealing with the assessed risk.

A parallel study has also been made by the United Kingdom Department of the Environment in conjunction with the Local Authorities concerned of the socio-economic, environmental and legal aspects of problem (policy considerations).

The data search and the technical appraisal to devise a risk strategy are described. Policy considerations arising from the existence of the mines are briefly outlined.

Introduction

The West Midlands of England, [Fig 1], was a leading area for iron making during the period of the Industrial Revolution between about 1780 and 1900, appropriately being known as the Black Country. Huge quantities of coal, iron-ore, fireclay and limestone were extracted from thousands of mines within the area shown in Fig 1, most of the minerals lying within 300m of the surface. All mining has now ceased within the Black Country, the last limestone mine closing in 1935.

The geological situation in the West Midlands [Fig 2] favoured easy exploitation, as thick seams of high quality limestone in Silurian strata crop out close to, or directly underlie abundant seams of coal and iron-ore deposited during the Carboniferous Middle Coal Measures period. Mine shafts originally sunk for coal and iron-ore were deepened to obtain limestone, thus providing all the necessary ingredients for iron making from the one mine.

There were four principal limestone seams, but only the Upper Wenlock and Lower Wenlock Limestones, respectively 6 to 7 and 9 to 10m thick, were extensively mined. The Aymestry and Barr Limestones were principally quarried at outcrop, [Fig 1].

It was general practice to abandon all mines as they stood, no attempts being made to backfill the cavities. The limestone strata are generally stronger than the overlying coal measures strata, and in consequence many of the limestone mine cavities have remained uncollapsed, whereas the coal and iron-ore mine cavities generally collapsed within a few years of being abandoned.

Whilst ground movement due to the collapse of mines has occurred from time to time over the past 150 years, it had been thought that the collapse

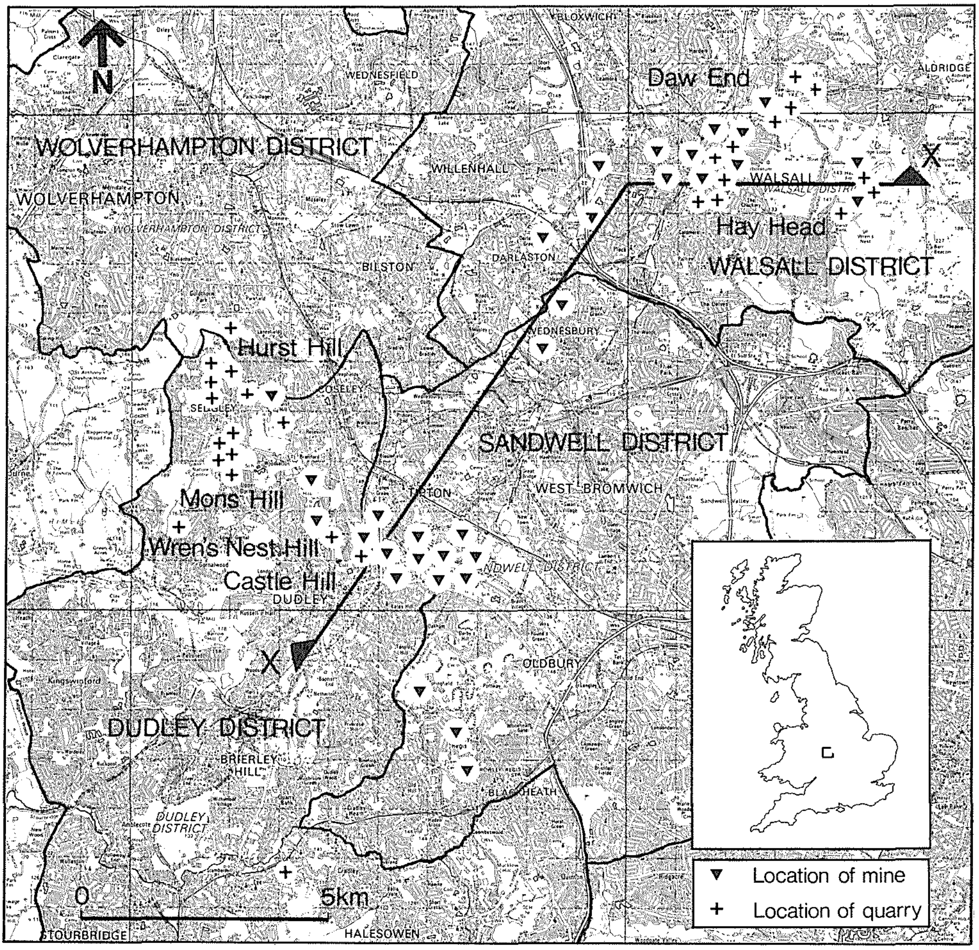

Fig 1. Plan of West Midlands showing limestone mines and quarries

of mines deeper than about 60m was unlikely to have any significant effect on the surface. However, in 1978, subsidence of 1.5m over a mine at a depth of 150m caused damage to industrial buildings in Sandwell. It thus became clear that deeper workings might not be as safe as had been thought, and that progressive deterioration in the condition of workings may lead to an increasing frequency of collapse and subsequent surface disturbance. The limestone mines were thus seen to present potential problems on a scale beyond the means of Local Authorities (LAs) or private landowners to remedy and assistance was sought from Central Government.

The LAs became concerned that they may be held to have been negligent and be liable for compensation for any damage. Many of the limestone mines underlie town centres and other developed land and thus large areas were being blighted, as approvals under the Building Regulations were not being given.

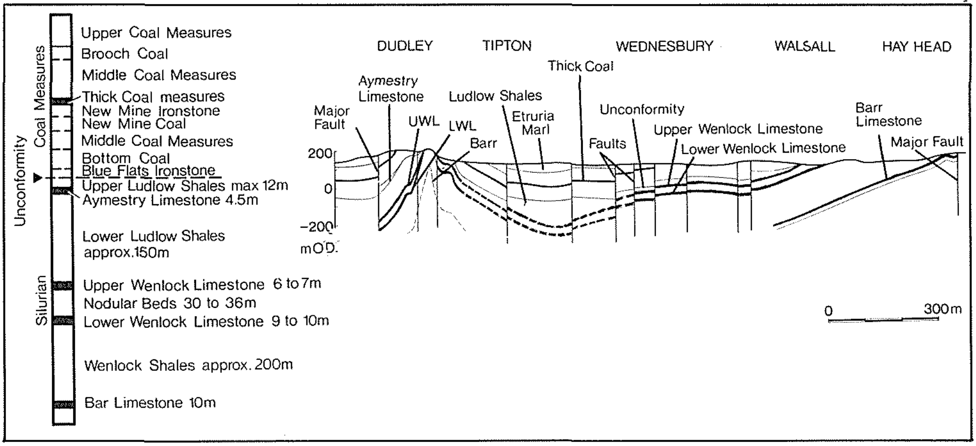

Fig 2. Geological column and geological section along line X-X

In recognition of this situation, the Government passed legislation to enable the grant-aiding of remedial works on land which is liable to become derelict, neglected or unsightly by reason of actual or apprehended collapse of the surface as a result of underground mine workings other than for coal.

The Department of the Environment (DoE), together with the LAs concerned, commissioned Ove Arup and Partners, consulting engineers, to assess the extent and degree of risk of surface collapse due to old limestone mines in the West Midlands and to consider what action was required (Ove Arup and Partners 1983a, 1983b). A parallel study was undertaken by the DoE in co-operation with the Metropolitan Borough Councils of Dudley, Sandwell and Walsall and the West Midlands County Council of the legal, socio economic and environmental aspects of the problem (Steering Group for the Black Country Limestone Study 1983).

The Ove Arup Study naturally divided into two separate but complementary activities, the search for data about limestone mines and the technical study.

Data search

The objective of the data search was to discover the extent of information concerning limestone mines that remains. Geological plans and detailed mine plans were produced and a comprehensive record of the majority of the information provided in the form of a data bank.

The quality and quantity of information on mines varied greatly. For mines worked before 1873, at which date the keeping of mine plans became mandatory, plans were generally made for the purposes of ascertaining the area of limestone removed, upon which payment of a fee or "royalty" would be due. If the mine was owned, there was no reason to prepare such plans.

After 1873 statutory plans were made; but it was not mandatory to deposit these with a central authority and over the years they have become widely scattered, and probably some have been lost. As an example, the only copy of the survey plan of a large limestone mine beneath a built up area, was found in an auction room in a town 50 miles from the West Midlands.

A few mines are still open and accessible and the accuracy of the mine plans can be checked. The majority of mines are flooded and the access

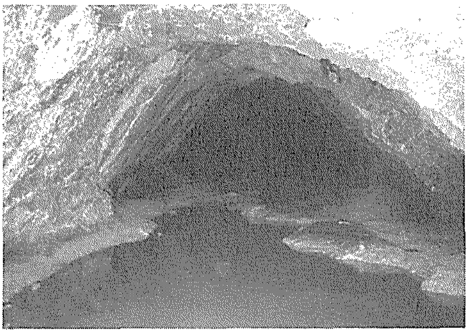

Fig 3. Gallery type mine

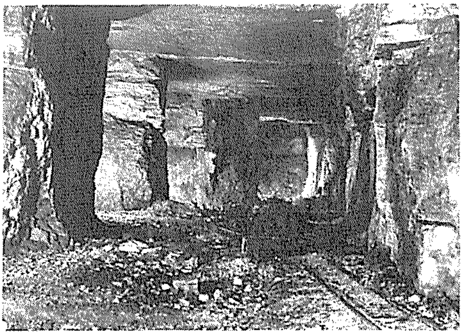

Fig 4. Pillar and room type mine

shafts are sealed, so there is no direct way of confirming the accuracy of the plans. Some plans are marked to show that they were made at the abandonment of the mine, whereas others appear to show only part of a mine, and the full extent of the mine cannot be ascertained.

In some cases the evidence for the existence of a mine is either a written description in a book or newspaper article, or an account of the financial transactions concerning a mine. Newspaper accounts generally record a mine disaster. In one example the account of a collapse of a mine, which gave rise to a large subsidence, also refers to the existence of an adjacent quite large mine for which only one other piece of written evidence has been found. No plans of either mine have been found.

The criterion applied during the data search was that if two or more independent pieces of information referred to the existence of the mine, its presence would be shown on the mine plans prepared at 1:2500 scale. In some cases the area of land owned or leased, below which the mine must lie could be identified.

Technical Study

The objectives of the technical part of the Study were to investigate the extent and physical characteristics of the limestone mines and the rocks surrounding them, and to establish the degree of risk, if any, of ground movements occasioned by collapse of the mines; also to consider and recommend what remedial and other works should be undertaken.

Initial appraisal

Four mines still open were inspected during the early stages of the Study, two in the Upper Wenlock Limestone and two in the Lower Wenlock Limestone. Two of the mines (one in each limestone) were of the "gallery" type, in which the mine took the form of a tunnel along the strike of the steeply dipping bed [Fig 3]. The other mines were of the pillar and room type [Fig 4], with pillars typically 8 to 15m square and 10 to 20m apart. It appeared that somewhat different processes of deterioration leading to collapse of the mine occur in the Upper, from those in the Lower Wenlock Limestone.

Crownhole subsidences

Falls of rock from the roof of the original mine are to be found in the mines in both limestones. In the case of the Upper Wenlock limestone, these roof falls expose the base of the Ludlow Shales, mudstones with little calcium carbonate content. These soften relatively rapidly and give rise to

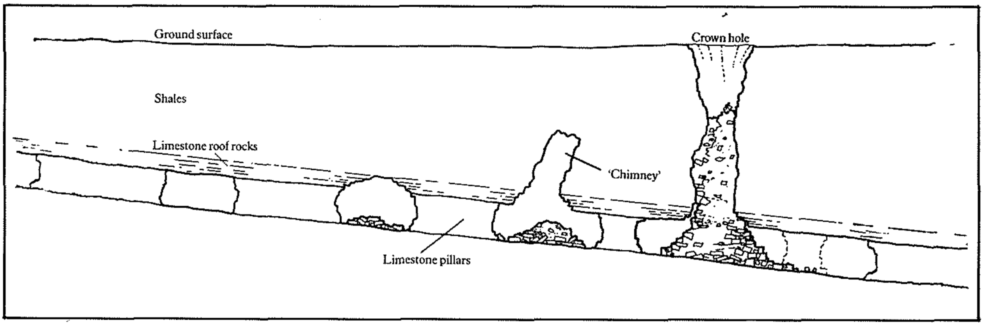

Fig 5. Collapse of shales into mine leading to "chimneys" and crown holes

chimneys with near vertical walls, and eventually to crown holes [Fig 5]. No instance was found of collapse of limestone pillars.

In contrast falls of the roof rocks of the Lower Wenlock Limestone expose the Nodular Beds, irregular limestones interbedded with mudstones. These separate along the mudstone beds when exposed to weathering, and roof collapse takes place bed by bed, forming corbels and resulting in shallow dome-shaped roof collapses. Unless faulted in such a manner as to precipitate collapse, Nodular Beds are unlikely to give rise to surface subsidence provided, firstly, that the major part of the full thickness of over 30m of the Beds are present above the limestone mine and, secondly, that the distances between sound limestone pillars has not been increased above about 40m by either pillar removal or pillar failure.

There are no records of crown holes from mines deeper than 70m.

Detailed investigation

In relation to flooded mines lying at depths greater than 50m, little previous fieldwork had been undertaken. Investigation was undertaken by drilling to obtain core samples, accompanied by in-situ and laboratory tests, and surveys of the drillholes and cavities using television cameras and ultrasonic scanners.

The mines selected for investigation included the Cowpasture Mine at Wednesbury Old Park, the site of the 1978 subsidence which caused considerable damage and gave rise to the Study. This mine in the Upper Wenlock limestone was worked between 1855 and 1879, and good quality mine plans are available.

It was discovered that the mine lay at a depth of about 150m and the limestone pillars have remained intact. However the Ludlow Shale stratum which formed the roof rock was found to be heavily fractured and the lower part has collapsed into the mine cavities. Above the limestone pillars the Ludlow Shales have closely-spaced shear surfaces, indicating that crushing of the lowest part of the Ludlow Shales over the limestone pillars occurred under the overburden pressure of about 2000 kN/m^2 (2.0 MPa).

Tests were made to measure the strength of intact Ludlow Shales and limestone, and the bulk strength of the rocks, taking account of joint spacing and inclination. The presence of mudstone-filled joints and other features, was assessed using a combination of the CSIR geomechanics classification system and the NGI tunnelling quality index (Hoek and Brown, 1980). The empirical failure criterion given by Hoek and Brown was used to derive the factor of safety of pillars of various width to height ratios [Fig 6].

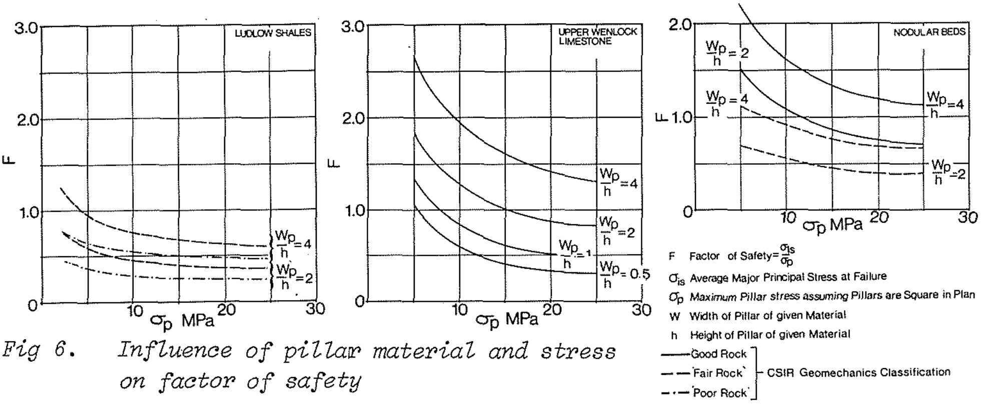

Fig 6. Influence of pillar material and stress on factor of safety

From Fig 6 it can be seen that limestone pillars, of "good" quality rock with a width to height ratio of about 3, were not likely to fail under an average pillar stress of 2.0 MPa.

However, the factor of safety of a pillar of Ludlow Shale will be less that 1.0 at a stress of 2.0 MPa for a rock condition assessed as "fair" or worse, and width to height ratio less than 4. The limestone pillar widths at the Cowpasture mine were of the order of 12 metres, so roof falls reaching only 4m above the top of the limestone stratum into the Ludlow Shales, were sufficient to bring about failure of the Ludlow Shales.

General subsidence

At the Cowpasture mine collapse of the Ludlow Shales into the mine cavities, brought about by crushing of the Shales over the limestone pillars, did not give rise to crown holes in the manner that was evident at shallower mines. Instead a general subsidence in the form of a large saucer-shaped depression took place (Fig 7). At the centre of the depression the total settlement was greater than 1.5m, badly damaging recently built single storey factory units. The rate of subsidence was slow at first, increasing to reach a maximum rate of about 50mm a day and then decreasing gradually over a period of several months. Good correlation was obtained between the predicted shape of the depression, using the National Coal Board Subsidence Handbook (1975), and the area of the mine for which the average pillar stresses were highest and thus most likely to give rise to crushing of the Ludlow Shales.

Mines in Lower Wenlock limestone

Similar investigations were made at Daw End and Littleton Street in Walsall where the Nodular Bed forms the roof rock to the Lower Wenlock limestone. At both mines the cavities were still open and ultrasonic surveys showed that the layout of the mine and the shapes of individual pillars were as recorded on the mine plans made when the mines were abandoned about seventy five years ago. The mines are 50 to 70m below the surface.

Some 3500 dwellings, valued at about £57 million, lie within the affected areas; most are in LA ownership (66%) and practically all the dwellings are in good condition. Industrial and commercial floorspace totalling 465,000m^2 is potentially affected, most (62%) being in Walsall, near the town centre. The current market value is assessed at £64 million, not including plant and machinery and making no allowance for compensation for possible disturbance and loss of profits. Much of the building stock is old and replacement

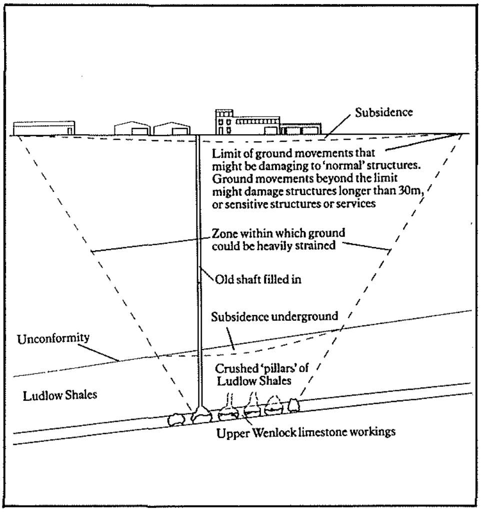

Fig 7. General Subsidence

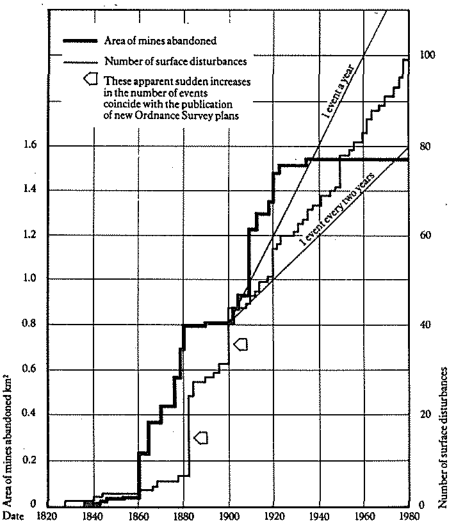

Fig 8. Area of mines abandoned and number of disturbances each year

value at current costs would be considerably higher. Amongst the properties affected are a number of important public buildings, including a hospital, two colleges, three schools, a bus depot and a parish church.

Constraints on the use of undermined land arise from the perception of the stability of the mines, the legal position of LAs in the exercise of their statutory functions, the landowners potential legal liability to third parties and the response to the existence of the mines by financial institutions, professional groups and the property market generally. The LAs had adopted a restrictive policy in respect of Building Regulations in limestone areas, hampering many proposals at a late stage and discouraging others at the outset.

Codes of Practice for Planning and Building Control have now been adopted which allow for the reasonable continuation of development in areas overlying or within the consideration zones of former limestone mines. However, there will still be sites where permission to develop will not be forthcoming and the costs of the necessary site investigations may be prohibitive for the smaller development.

The report of the Ove Arup Study and the Steering Group Report are available to all. As a result, landowners have claimed to have difficulty in selling properties in limestone areas and even when successful, only at a reduced price. Difficulties in obtaining planning permission or in demonstrating compliance with the Building Regulations may compound the problem and potentially developable sites may remain undeveloped.

Future action

The UK Government has set aside £1 million in the current financial year to begin work on the installation of monitoring equipment, site investigations and remedial measures to preserve surface stability. A further £2 million has been set aside for 1984-85 and the Government will continue to make provision for such work in future years. It is clear, however, that the problem will not be solved overnight and that a long-term programme of action will be needed.

The options for future action as indicated in the Risk Strategy [Fig 9] are:

(i) minimum work, comprising measures to prevent public access to mines and to fill in crown holes and subsidences as they occur;

(ii) monitoring of movements in the ground and of the condition of the mines to identify the onset of disturbance being transmitted to the surface and to review the likelihood of such disturbance;

(iii) investigation of the underground voids and the ground above them to prove their existence and extent and to determine their condition so as to enable the likelihood of surface disturbance to be assessed;

(iv) treatment of the mine to preclude the possibility that future surface disturbance may cause harmful damage to structures or services, by collapsing, strengthening, excavation or filling the mine;

(v) treatment of structures and services so that they can either withstand the effects of subsidence (retain serviceability) or allow safe evacuation of occupants (maintain short term integrity).

The height of the worked cavities was found to be about 7m and in places the mine roof had collapsed leaving a pile of rock debris on the original floor. Roof falls were found to extend only a few metres above the roof of the original mine cavities.

The higher strength assessed for the Nodular Beds, with the rock classified as "good" and fair", as opposed to the "fair" or "poor" for Ludlow Shales, is reflected in the higher factors of safety for given pillar width to height ratios [Fig 6]. Crushing of the Nodular Beds leading to surface subsidence will occur only if the pillar width is smaller than 5m. The mine plans show that there were in places at the Daw End mine pillars with widths less than 5m, and some crown hole type subsidences have occurred in the ground above such places.

At Littleton Street there was a period about 1860 when the pillars in one part of the mine were "robbed", so reducing the widths that they collapsed and a very large crown hole type subsidence took place.

Risk strategy

The investigations confirmed a clear distinction between the modes of collapse in Upper and Lower Wenlock Limestone mines. There was however insufficient information concerning the precise circumstances leading to subsidences. It was thus not possible for a risk strategy involving all 31 known mines to be based upon appraisal of the physical parameters.

A statistical approach was therefore adopted, taking into account those features that could be ascertained reasonably accurately for all mines.

For most mines the limestone stratum mined, the dates between which mining took place, the depth of the mine and the mine layout can be ascertained. As shown by the graph [Fig 8], mines were abandoned at a more or less uniform rate between 1850 and 1920, the rates being slower before 1850 and after 1920, the last mine closing in 1935. During the period 1850 to 1920 the rate at which subsidences occurred was also a more or less uniform rate, about one surface disturbance (either a crown hole or a general subsidence) every 1½ years. Since then surface disturbances have continued to occur at various mines throughout the area at about the same rate as during the period of mining.

The risk strategy at present being used is shown diagrammatically in

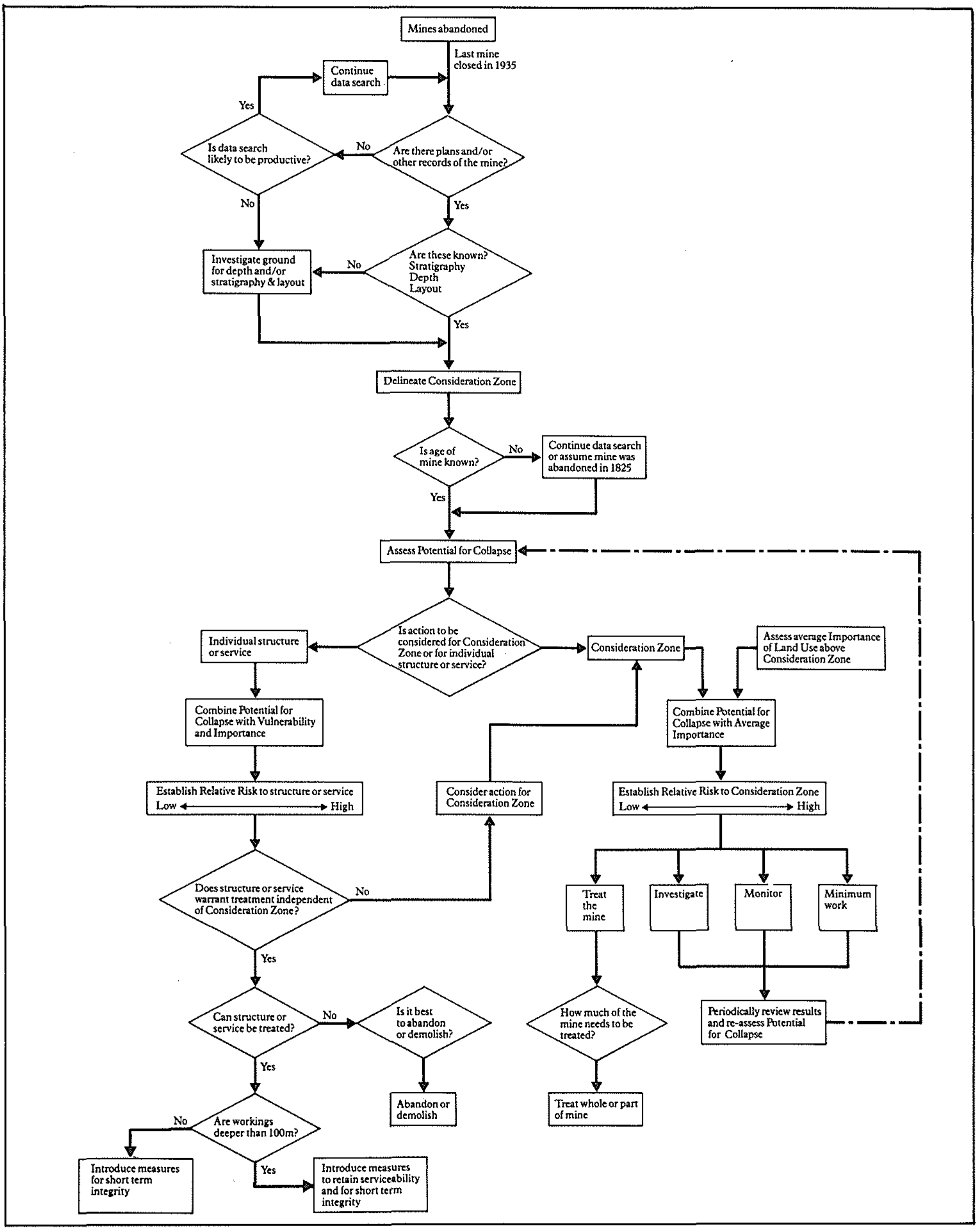

Fig 9. Risk strategy

Fig 9. Certain key words and phrases require explanation:-

- Consideration Zone; the area above a mine within which structures and services are likely to suffer more than minor damage due to subsidence; and thus consideration should be given to the need to investigate the ground in relation to surface movements that could be caused by abandoned limestone mines.

- Potential for collapse; the relative likelihood of subsidence occurring above a particular mine during the next year.
- Importance; the results of evaluating the effect of a subsidence on the community, essential facilities being heavily weighted.
- Vulnerability; the ability of a structure or service to accommodate subsidence

Policy Considerations

Of the 300 km^2 in the West Midlands beneath which limestone exists at a depth at which it might reasonably have been worked, only about 486ha is affected by abandoned limestone mines. This area includes the surface area of all known mines together with the consideration zones around those mines, and areas where mines are suspected to exist but the precise boundaries can only be defined in terms of the land which was leased for limestone extraction; the latter are confined to Walsall and comprise about 63% of affected land in that Borough.

Within the total area affected, there are estimated to be about 12,000 jobs, a resident population of about 9,000 and property valued at about £152 million. Only 22% of the affected land is unused or in agricultural use. Much of the land lies in or close to town centres and older urban areas and residential (19%) and commercial and industrial uses (25%) form a significant proportion of the total area. The extent of recreational land (19%) is unusually high, partly reflecting decisions to use undermined land for such purposes.

The options chosen for each area will depend on the perceived level of relative risk of ground movement and the likely effects of such movement on land use and surface structures. The DoE and the LAs with Ove Arup and Partners as consultants are now working together to establish a clear order of priorities for future action. In the first instance, priority has been given to detailed investigation of certain mines to resolve uncertainties as to their extent and condition, monitoring of ground movements and microseismic monitoring of activity in the mines. A major task for the summer of 1984 is the testing in the field of a novel method of mine filling by injecting through boreholes a thick paste of waste rock material using conventional concrete pumping equipment. Meanwhile, consideration continues to be given by all parties in central and local government, public utilities and the private sector to the various issues arising from the existence of the mines and to assessing the priorities and options for the 31 mine sites identified.

Acknowledgements

The engineering Study was guided by a Steering Group comprising members of the Department of the Environment and representatives of the Metropolitan Boroughs of Dudley, Sandwell and Walsall, and the West Midlands County Council, who also undertook the study of Policy Considerations. The views expressed are those of the authors alone, and do not necessarily represent those of the DoE and LAs concerned. The ready co-operation of individuals and departments represented on the Steering Group, and the help given by the Institute of Geological Sciences, libraries, museums, local engineers and historians is gratefully acknowledged.

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GROUND MOVEMENTS AND SUBSIDENCE IN NIGERIA'S COAL AND METAL MINES

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Abstract

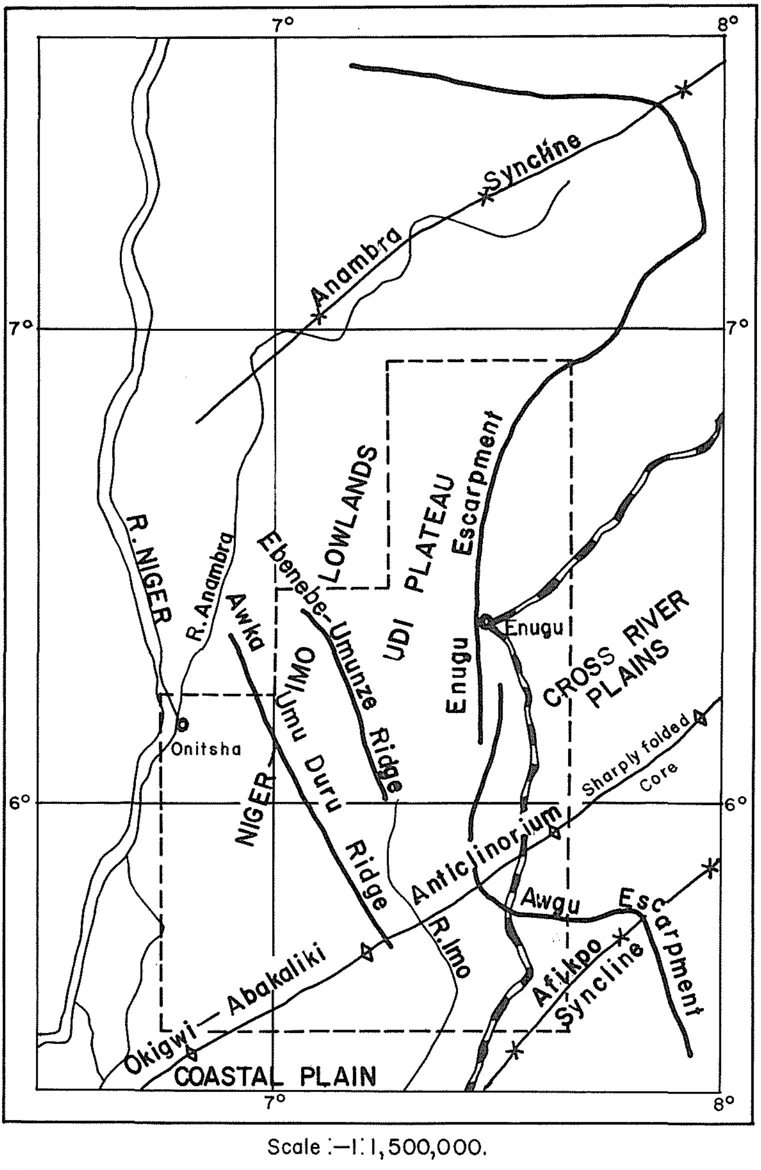
Ground movements and subsidence are in various forms, resulting from the uncontrollable tectonic to controllable human activities. Mining and quarrying industriés have been identified as the contributory human activities, hence the scientific and technological research programmes by nations to combat the adverse environmental effects and residual hazards to public health.

Nigeria with her coal and metalliferous mines is facing two major hazards of surface subsidence from underground coal operations and ground movements and erosion generated by the unrehabilitated metalliferous mines in Plateau and Bauchi States of the Federation. Her restoration and reclamation schemes though laudable, are colossal and expensive and are not capable of completely alleviating the destruction from alluvial operations. There is the great need for research into the social and economic development of the affected areas.

General

Ground movements and subsidence are regular occurrences in many parts of the world. They are in various forms, ranging from the violent volcanic eruption to an induced "creeping" in coal mining or fall of ground from blasting operations. But perhaps man's great concern about some of these movements is their destructive nature, which outweighs their value, if ever any has been considered valuable, the fact that tectonic activities have been associated with magmatic mineral deposits notwithstanding. In addition to tectonic activities, human activities also contribute immensely to the ground movements and subsidence, such that are now occupying the minds of all participants to this seminar. While human activities are controllable, thereby reducing or eliminating their damaging effects, tectonic activities have involved nations in scientific and technological research programmes. Let us hope that these programmes will in no small measure lead to remedies that will reduce the disasters to which mankind has been exposed. It is my intention at this juncture and from now on to concentrate on those human activities, that had contributed and are still contributing to ground movements and subsidence. These activities have been identified in mining industry, an industry whose value to development and civilization cannot be over-estimated. It is in this regard that every legislation directed towards the protection of man against the ills of this industry, such as adverse environmental effects, and residual hazards to the public health, does not fail to declare that the extraction of minerals is essential to the continued economic well-being of the State and to the needs of the society. Consequently such legislations invariably aim at:-

1. Preventing or minimising adverse effects on the environment and protecting the public health and safety.
2. Protecting the surface against subsidence, reclamation of mining lands



Scale :—1:1,500,000.

FIG.1 - Part of Southern Nigeria Showing relief forms and Principal fold axis.

to permit the continued mining of minerals, and the protection and subsequent beneficial use of the mined and reclaimed areas.

It is a well-known fact that there is probably no industry that depends upon so great a variety of other arts and that involves so many branches of science as does mining. Nevertheless, the actual extraction of the mineral must always remain the most important task of the mining engineer, and the manner in which this is conducted necessarily depends upon the mode of occurrence of the mineral itself. Furthermore, in working any mineral deposit, however, we are advised to have economy, efficiency and safety as the ruling watch-words, as they go together. Safety should be taken to cover, not only the prevention of accidents, but also the removal of all that is injurious to human life and health, of everything that reduces human efforts, so that it should become the greatest factor in economical production. It is in this vein that the effects of mining industry on the safe economic existence of the teeming population of farmers within the mining districts, as well as the general interests of the public, have become a matter of great concern to the Nigerian Government. Endowed with both igneous and sedimentary mineral deposits, to which all known methods of exploitation, both new and old are applicable, none of these methods has been a subject of any research programme in Nigeria.

Nigerian Coal Mines: How prone to subsidence:

Let me first of all review the topographical, geological and structural characteristics of Nigerian Coalfield, with emphasis on the seams being worked by the Nigerian Coal Corporation (N.C.C.) (Fig.1).

Stratified deposits are known to have aqueous origin, which accounts for many of their characteristics, which in turn have an important bearing on methods of working. Having been deposited underwater, they tend to lie horizontally, but this is subject to many eventualities. Subsequent igneous action and movements of the earth's crust have tilted and bent those deposits to all angles and curves, causing numerous faults and dislocations, which are so problematic in underground mining. Nigerian coal deposits are no exception.

The muds, silts and sands comprising the lower coal measures were found by geologists to have been deposited in extensive stretches of shallow, fresh water. Shoaling of the basin of sedimentation took place at intervals, and some areas were converted into swamps, covered by thick vegetation which has given rise to coal seams. The main coal fields are at Enugu. Enugu on the other hand is situated on the western edge of the Cross River plain and is dominated by the Enugu escarpment just west of the town itself. For the first 122m - 152m, the escarpment is steep, but it then rises more gently to above sea level to about 183m above Enugu. Further-west several large but low hills attain an elevation of nearly 518m. The escarpment is much indented by deep valleys which have been cut by head-waters of Ekulu, Nyaba and Atafo streams. The plain on which Enugu stands is underlain by Enugu shales, which occupy a wide belt of country at the foot of the escarpment. The lower coal measures outcroping in the streams on the lower slopes of the escarpment west of Enugu, between the Asata and Ekulu Rivers, comprise of alternating sand-stones, shales, sandy-shales and mudstones with coal seams or carbonaceous shales. In this area there is a characteristic pattern in the sequence of rock types, marked by units which are repeated at least five times, which suggests repeated uplift and subsidence of the land during deposition. A typical unit is as follows:-

5. Shale or sandy shale, etc.
4. Coal, sometimes shaly at top.
3. Carbonaceous shale passing downwards into shale.
2. Sandstone with a few shaly layers or alternating sandstones and shales.
1. Shale or sandy shale.

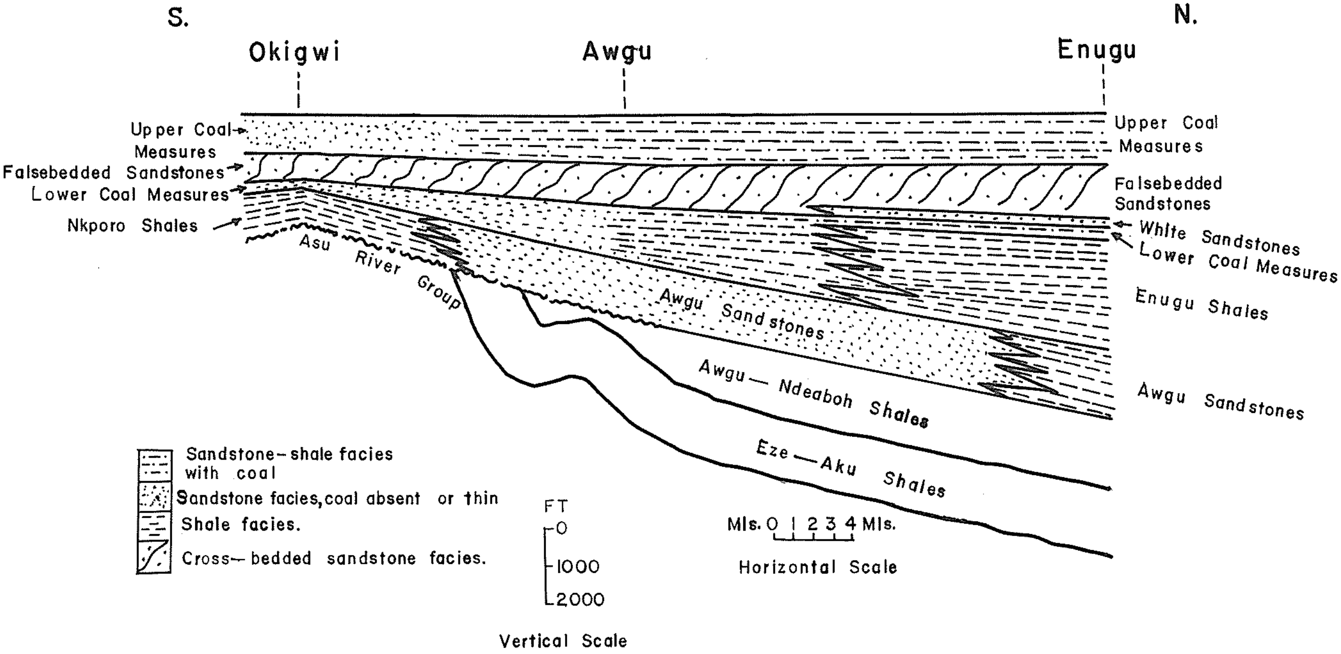


FIG.2 Facies - Changes over the Okigwi axis.

The sandstones in each unit vary considerably in thickness and sometimes rest directly on the coal (Fig.2).

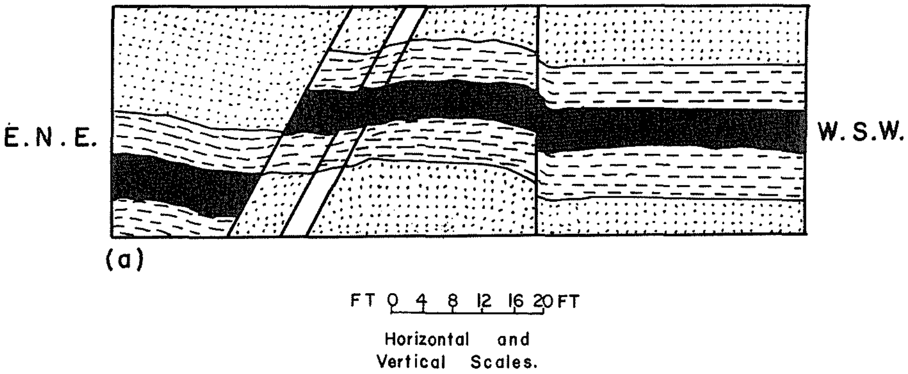
Methods of work

That the actual extraction of the mineral must always remain the most important task of the mining engineer, and that the manner in which this is conducted necessarily depends upon the mode of occurrence of the mineral, are fully reflected in the nomination or adaptation of methods of work, to cope with the characteristics of Enugu coal field. In these methods, the principle that removal of a stratified deposit from the crust of the earth leaves an empty space and sets free-force in the surrounding strata, which brings pressure and strain on the roof and sides and floor of all open excavations was acknowledged. Consequently in developing the No.3 seam early in 1915, adits were sunk, working faces were adequately provided, the road ways for the transport of mine products and materials and of men between working faces and adits were constructed.

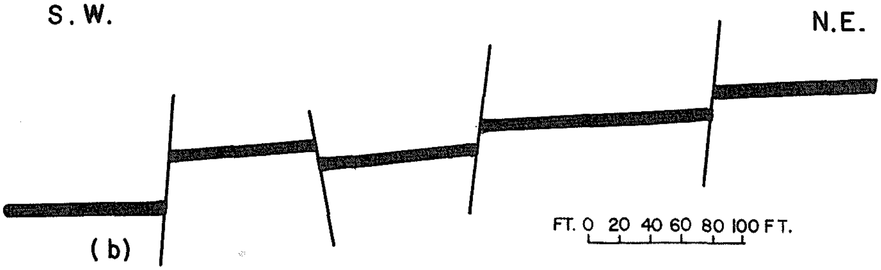
Six faults or fault systems were however associated with the field namely:- Nyaba fault, Hayes fault system, Iva fault system, Ogbete (Obwetti) fault system, Obuga fault and Juju Hill fault (Fig.3 (a) (b) (c)). These of course indicate some complications which had to be coped with. However as a result of the favourable situation of the strata dipping to the west, or west-north-west at a low angle, generally between 1° and 3° into the steeply rising escarpment, it has been possible so far to mine the coal from adits driven into the hill-side. Mining was by the pillar and stall method. The seam was worked "on bord" with the face parallel to the cleat, thus facilitating the breaking off of large slabs. Stalls were driven 1.8m wide and crossed one another at right angles to develop pillars 27m x 17m. The dimensions of the pillars were however varied to suit prevailing conditions at different places. The roof was first supported with local timber and later with hardwood (mangrove). It is pertinent to state that, in those early days mining was by the crude method of hand-hewing of the solid ore and evacuation was by head loads using baskets. The roof of the seam was strong and stood up well for long periods without support. As time went on, mechanization followed gradually. Explosives were introduced in 1944 with consequent reduction on stress and strain on hand-hewing. Coal cutting machines followed, and conventional long wall system was introduced about 1952 at Hayes Mine (now Okpara Mine). Face lengths varied from 45m to 100m. Complete mechanization has now been effected using shearer loaders, armoured chain conveyors, belt conveyors and power supports. The wastes were controlled with strip packs. Total extraction of coal was aimed at, achieving about 98%. During robbery the pillars were breasted out towards the waste.

Under the above conditions the Nigerian Coal Corporation has operated the following collieries:-

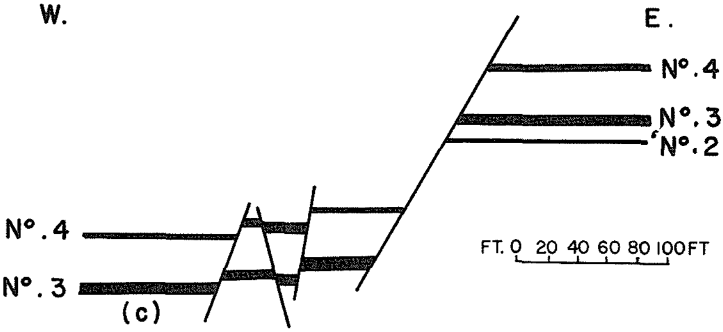
1. a. The first Udi Mine 1915/1916 - closed. (b) Iva Mine, opened 1917, now closed. (c) Hayes Mine (now Okpara Mine) opened 1957, still operating.
   d. Ekulu Mine (now Onyeama Mine) opened 1956, still operating.
   e. Ribadu Mine, opened 1961, now closed.
2. These Mines (Fig.4) are relatively shallow, being and having been operated at an average depth of 213m.
3. All operations were and are still under extreme wet conditions, with spasmodic flooding.
4. Underground water has been found to be acidic.
5. Most of the mines have serious problems of strata control.



Obwetti fault system as formerly exposed in the main drift of the old Udi Mine.



Iva fault system on N° 6 road leading to 14 West District, Iva Mine.



Iva fault system exposed in the Conveyor Drift, Obwetti Mine.

FIG.3 (a) (b) (c) - Fault Systems.

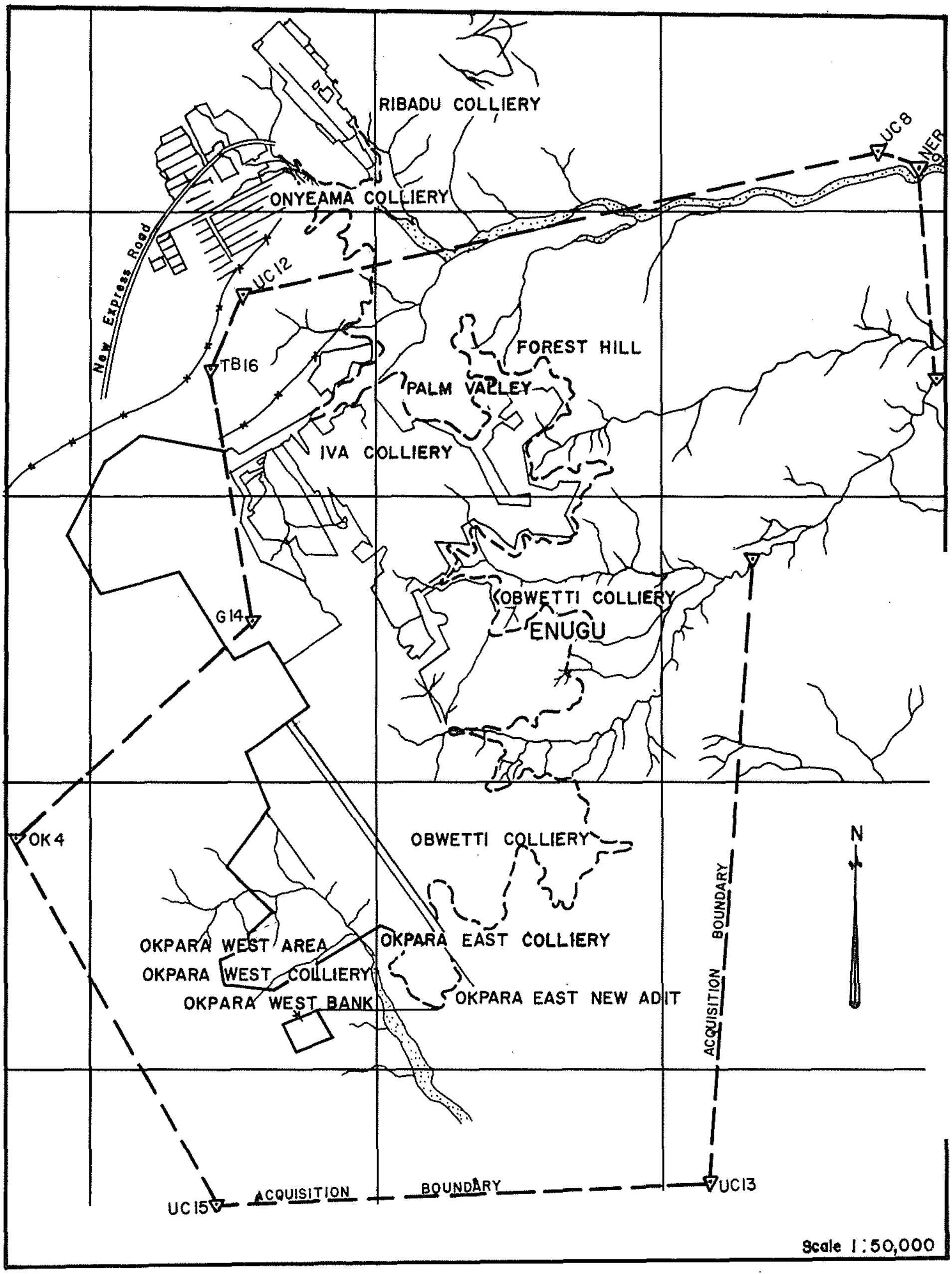
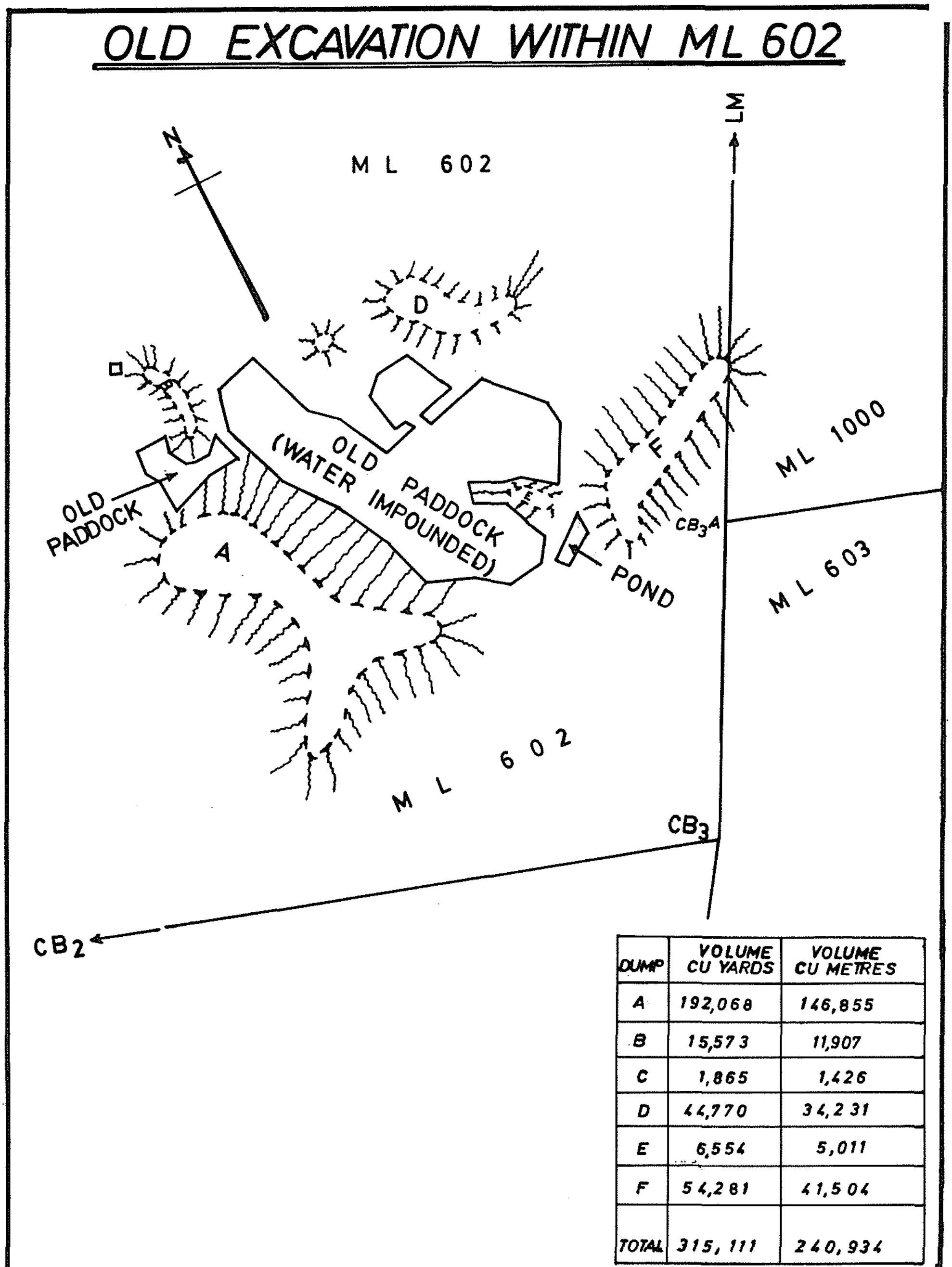


FIG.4 Advance of workings to-date, N.C.C. Collieries Enugu.

The major questions here are:-

1. How effectively had the goaf been restored or filled as its area extended, considering the method of work?



DUMP	VOLUME CU YARDS	VOLUME CU METRES
A	192,068	146,855
B	15,573	11,907
C	1,865	1,426
D	44,770	34,231
E	6,554	5,011
F	54,281	41,504
TOTAL	315,111	240,934

FIG. 5 :– Old Excavation Within M. L. 602

SCALE :– 1 : 4,000'

FIG.5 - Old Excavations within Mining Lease No. 602

2. With what materials was the goaf filled and what is the compressibility of such materials?
3. How were the fractures and subsidence of the overlying strata controlled?
4. Has the movement of ground within the abandoned mines stopped?
5. What are the effects of the various fault systems?
6. What are the effects of flooding on the stability of the floor and supports including any permanent pillars?
7. How many cases of surface subsidence have been reported and handled? Is it safe to assume that since the adits and workings are within Udi hills, ground movements within the abandoned workings will not be transmitted from these workings to the surface?

The fact is that despite the provisions of the law, no detailed study has ever been made of the characteristics of subsidence in Enugu Coal Fields, even though there had been evidence that fractures associated with subsidence reached the surface only in areas where pillars had been left. The following facts are in the circumstances noteworthy:-

a. It is impossible to prevent movements of ground and subsidence altogether. (b) The impracticability of absolutely filling up of an excavated area is not contestable, especially in flat seams, one of the problems in filling being that although an opening may technically be completely filled, in practice there will be a smaller mass of material in it, the remainder of the space being taken up by the voids between the particles. This means that there will remain the problem of significant subsidence even in a filled mine, though subsidence will be reduced. If, however, a suitable and economic cement can be developed, the value of fill as a support in worked-out areas can be enhanced further. In this connection the reported development of such a cement at the British National Coal Board should be encouraged since research work on waste fill could result in deeper mining being achievable and improvement of environmental control of all underground mines, at the same time reducing the surface environmental impact with regard to spoil tips and subsidence.

c. In realisation of the serious effects of surface subsidence due to mining, Nigeria's Safe Mining Regulations provide for the protection of the surface where mining operations have caused subsidence cavities on the surface, or where such are likely to occur. Such places should be securely fenced in and conspicuous notice boards inscribed 'WARNING' erected and maintained as long as there is danger. For the protection of ground and any surface objects which it is necessary to protect in the interest of personal safety or public traffic, and the removal of which may be inexpedient, the reefs, coal beds or other mineral deposits shall be left intact not only vertically below the same, but also for such a distance beyond as the Inpector incharge of the Inspectorate may consider necessary. Permission for the entire or partial excavation of the ground beneath such surface objects may be obtained from the Director of Mineral Resources to the extent and under such precautions and conditions as he may prescribe in each separate case. Furthermore, the driving of tunnels not exceeding 1.8m in width through such safety pillars for the purpose of connecting two separate mines or parts of mine may be allowed with written permission of an Inspector, upon precautions prescribed by him being observed.

Metalliferous Mining

Nigeria's main metalliferous alluvial mining field is within the two adjoining states of Plateau and Bauchi. The industry is now over 81 years old. Firewood as a source of energy, was the earliest form of fuel used when mechanised tin mining operations commenced at the turn of the century.

By 1920, internal combustion engine utilizing oil and petroleum as fuel were introduced to augment steam power. The use of hydro-electric power for mining did not take off until 1929/1930, when the Nigerian Electricity Supply Company Limited was set up to develop hydro-electric power available from the Kurra Falls. The system's capacity was raised from the original 5.5MW to 6.0MW in 1930, and by 1936 a further 4000KW of plant was installed at Kurra Falls Station, bringing the system's total capacity to 10,000KW. It was therefore possible for mining industry to adopt electro-mechanical and hydraulic mining methods involving draglines, electric and diesel, dredges, gravel pumps, shovels, monitors and elevators and all shades of earth-moving equipment, Mining consequently progressed from labour intensive open-hand-paddocks and tributing to fully mechanised paddocks, the overall effects of which were:-

a. Deep and wide excavations, which in various locations went beyond water tables.
b. Stacks of overburden from these excavations became man-made-hills within every mining district.
c. Large ponds in many locations became a permanent feature; and agricultural lands became scarce, while inhabitants were resettled in non-mining districts.

The non-inclusion of restoration clause in her first mining law gave rise to this unsatisfactory situation. It was not until 1945 that the situation was rectified in the revised legislation.

It is noteworthy however, that while the question of subsidence over coal mines remains speculative, the Government has since 1946 and 1949, embarked on restoration and reclamation programmes respectively. Reclamation in its true context means the combined process of land treatment that minimises water degradation, air pollution, damage to aquatic or wildlife habitat, flooding, erosion, and other adverse surface effects, incidental to underground mines, so that the mined lands are reclaimed to usable condition, which is readily adaptable for ultimate land uses and create no danger to public health and safety. The process may extend to affected lands surrounding mined lands and may require backfilling, grading, resoiling, revegetation, soil compaction, stabilization or other measures depending on the end use of the land so reclaimed. Though laudable, it is a colossal and expensive programme. A typical reclamation area is shown in Fig.5.

It will be observed from the figure that absence of dumps renders reclamation impossible or unduly expensive because of the haulage of overburden from a new distant source.

With regard to restoration, the Minerals Act permits the Minister in his discretion to impose restoration conditions on any mining land granted by him. One effect of the extensive pre 1945 mining operations was the scarcity of agricultural lands for local inhabitants to the extent that further alienation for mining or other purpose meant insufficient land remaining for safe and economic existence of the people. There was therefore the need and necessity to protect the occupiers of the land and at the same time, in the general interest of the country to ensure that as much mineral as possible is won.

The amount of restoration imposed on any lease is based on the approved method of mining namely:-

1. Tributing, Lotoing and Underground mining	100 %
2. Gravel Pumping, Ground Sluicing and Hand-paddocking	80 %

3. Use of Dragline, Diesel Earth-moving Equipment and Dredging	75 %
4. Use of Hydraulic Elevator	70 %

No matter how practicable these conditions may appear to be, it is most improbable that the farm lands mined could be restored to a condition not inferior to that which was there before mining operations. It therefore implies that neither the reclamation programme nor the restoration scheme is capable of completely alleviating the destruction to surface resulting from alluvial mining methods. This is all the more reason why research into the social and economic development of these mined areas is strongly recommended.

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LAND SUBSIDENCE IN NATURAL GAS FIELDS

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Abstract

The extraction of water containing natural gas causes land subsidence, and so, houses, roads, bridges etc. are often damaged. The purpose of this report is to show the mechanism of land subsidence and to establish an analytical method for its prediction in natural gas fields.

The prediction method of subsidence mainly consists of three parts:-

1. Determination of the change in the piezometric head with time at each well
2. Elaboration of the plan distribution of the piezometric head in the gas seam by the finite element technique
3. Calculation of land subsidence, on the assumption that the gas seam behaves as an elastic material

Introduction

Natural gas reserves in Japan are estimated to amount to about 400 billion m^3. The regular development of natural gas began in the 1930s. At present, it is being tapped in different places, such as Hokkaido, Chiba, Niigata, Miyazaki, Okinawa etc.. The extraction of water containing natural gas in these areas has caused land subsidence, with inherent damages to houses, roads, bridges etc..

The analytical examples in previous works based on the well known Terzaghi's consolidation theory were almost all related to cases of relatively shallow or unconsolidated strata, such as soft clay. But, for relatively deep or compact strata, account must be taken of the ground elastic properties. In other words, we must consider two different types of subsidence; the one which is plastic, is not recoverable, and the other, elastic, which is partially recoverable. This will lead to an understanding of ground rebound due to the control of discharge rate.

Kamata et al.(1976) analyzed the land subsidence in Funabashi gas field with the vertical two dimensional multi-aquifer model. Their procedure was based on the finite differential method and Hooke's law, with which only the subsidence profile of a particular section in the gas field can be obtained, but not the plan contour map of subsidence .

The purpose of this study is to show the mechanism of subsidence and to establish a method for its prediction in natural gas fields, especially for deep and relatively compact strata. Our prediction method is applied to a certain gas field, and the calculated plan contour map of subsidence is compared with in situ measurements.

Mechanism of land subsidence

The land subsidence in natural gas fields depends on the decrease in piezometric head in the gas seam, due to the extraction of water containing natural gas.

Fig.1 shows schematically the piezometric head around a well. When the water is pumped out from a well, the piezometric surface, depending on the permeability of gas seam and discharge rate, is lowered. In a three dimensional representation the piezometric surface is funnel shaped, and its outer limit defines the well's area of influence.

The relation between total pressure σ' and neutral pressure σ_n on solid substance is shown by Terzaghi as;

$$\sigma' = \sigma - \sigma n \qquad (1)$$

Where σ': effective pressure

Eq.(1) means that the decrease in piezometric head (neutral pressure) is equal to the increase in effective pressure, while the total pressure on solid substance of the ground remains constant. The land subsidence is caused by this effective pressure.

The amount of subsidence is proportional to the increase in effective pressure, or the decrease in piezometric head in the gas seam. For the purpose of predicting land subsidence, it is necessary to know the distribution of piezometric head in the gas seam, and also the properties of the ground especially, permeability, compressibility etc..

Analytical method

The analytical method we have proposed in this report mainly consists of three parts.(See Fig.2)

The first part is to determine the change in piezometric head with time at each well, in accordance with the well-known Theis equation for an unsteady flow to a well.

Seepage analysis by the finite element technique constitutes the second part. In this part, we can obtain the plan distribution of piezometric head in a gas seam.

The last part deals with the calculation of land subsidence. This is done by the above calculated values of piezometric head, on the assumption that the gas seam behaves as an elastic material.

The nonequilibrium equation for flow to a well as;

$$\Delta h = \frac{Q}{4\pi km}\int_u^\infty \frac{e^{-u}}{u}\, du = \frac{Q}{4\pi km} W(u) \qquad (2)$$

where $u = \dfrac{r^2S}{4kmt}$

$$w(u) = -0.5772 - \ln u + u - \frac{u^3}{2\cdot 2!} + \frac{u^3}{3\cdot 3!} - \cdots$$

Δh : decrease in pizometric head

Q : discharge rate

k : permeability of aquifer

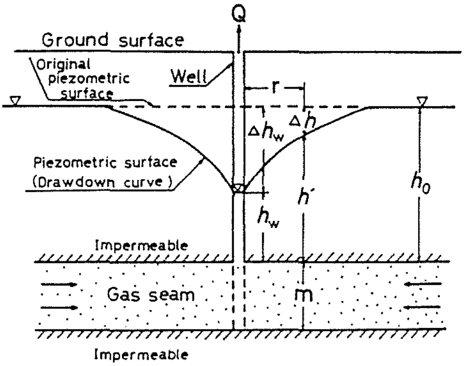

Fig.1 Schematic diagram showing variation in piezometric head due to the extraction of water containing natural gas

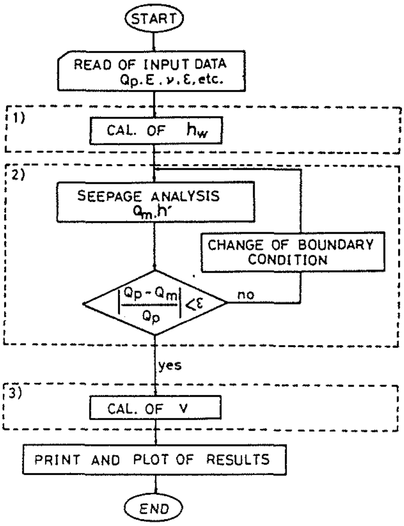

Fig.2 Flow-chart for calculation of subsidence

m : thickness of aquifer
S : storage coefficient
t : time
r : distance from well

Eq.(2) is known as the nonequilibrium, or Theis equation. If r is small and t is large, the series terms in Eq.(2) becomes negligible except the first two terms.

The drawdown head can be rewritten as

$$\Delta h = -\frac{Q}{4\pi km}\left(0.5772 + \ln\frac{r^2S}{4kmt}\right) \qquad (3)$$

Eq.(3) gives Δh under the constant discharge rate Q. If Q varies with time, the drawdown head can be calculated by the principle of superposition. For example, if a well discharges at a rate of Q_1 in t_1 period; and at a rate of Q_2 in t_2 period, the drawdown head at well can be written as

$$\Delta h = -\frac{Q_1}{4\pi km}\ln\frac{t_2}{t_1+t_2} - \frac{Q_2}{4\pi km}\left(0.5772 + \ln\frac{r^2S}{4kmt_2}\right) \qquad (4)$$

If $t_1+t_2=t$, and $Q_1=Q_2=Q$, Eq.(4) is naturally equal to Eq.(3).
The model in seepage analysis is a two dimensional confined steady state flow through a gas seam. It is assumed that the gas seam is fully saturated with water containing natural gas, and that Darcy's law holds good.

Hence, the basic governing equation is expressed as:-

$$kx\frac{\partial^2H}{\partial x^2} + ky\frac{\partial^2H}{\partial y^2} + Q = 0 \qquad (5)$$

where kx, ky : permeability in x, y direction, respectively
H : piezometric head
Q : discharge rate

We solve Eq.(5) by the finite element technique. Boundary conditions are ordinarily taken as follows;

1. Head or potential boundary condition
2. Flow boundary condition

In this report, we give the respective piezometric heads at each well and at the outer boundaries. For the piezometric heads at each well, we use the values calculated by Eq.(5), but taking special account of heads at the outer boundaries.

In the first step of calculation, we assume that the piezometric head is equal to the ground surface. Then the total calculated discharge volume Qm and the measured total volume of water Qp in the gas field are compared.

$$\frac{Qp - Qm}{Qp} < \varepsilon \qquad (6)$$

If Eq.(6) is not satisfied, the piezometric head at the outer boundaries automatically uniformly rises or decreases. That is, if Qm>Qp the piezometric head will make down and if Qm<Qp the piezometric head will rise.The calculation is iterated until Eq.(6) is satisfied.
The calculation of land subsidence based on the generalized Hooke's law is performed. In other words, we assume that the gas seam behaves as an elastic material.

The increase in effective load causes deformation of the gas seam, so the deformation appears as land subsidence.
We then calculate the land subsidence by the following equation.

$$s = \alpha \frac{m}{E} (1 - 2\nu) \gamma g \, \Delta H \qquad (7)$$

where s : subsidence
α : coefficient of subsidence
m : thickness of gas seam
E : Young's modulus of gas seam
ν : Poisson's ratio of gas seam
γg : unit weight of water containing natural gas
ΔH : decrease in piezpmetric head in gas seam

Eq.(7) is for a single gas seam. If there are multiple gas seam, we must sum up the subsidence of each gas seam.

In this report, we do not consider the horizontal ground surface movement; only the vertical movement is calculated.

Practical application

We applied our analytical method to one of the Japanese gas fields. This gas field has about 30 wells in about a $25km^2$ zone, and two gas seams. The depth goes from 400-1300m. Almost all the gas seams in this field are of Pliocene age. The gas seams have an inclination of about 5-10°, but in our analysis we assume they are horizontal.

Table 1 shows the input-data for analysis, and these data are deduced from laboratory tests on boring cores, and in situ pumping tests. Fig.3 shows the results of consolidation tests and uniaxial compression tests on boring cores at different depths in this gas field. In almost all the data, the yield stress Ys falls within the overconsolidated state, so we can expect to have the ground elastic property. And, the mean compressive strength at depth of 1000m is about 180 MPa. The relation between compressive strength Sc and Young's modulus E is shown in Fig.4, and E is roughly proportional to Sc, that is, E=120Sc. Fig.5 shows the change in piezometric head with time at a well. It is found that the result from Eq.(3) and

Table 1 Input-data

Seam	E(GPa)	ν	m(m)	k(cm/sec)
A	2	0.25	240	$4x10^{-5}$
B	3	0.25	480	$4x10^{-5}$

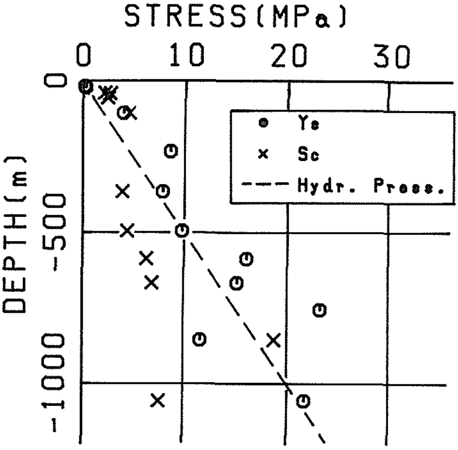

Fig.3 Results of consolidation and uniaxial compression tests with boring cores at different depths

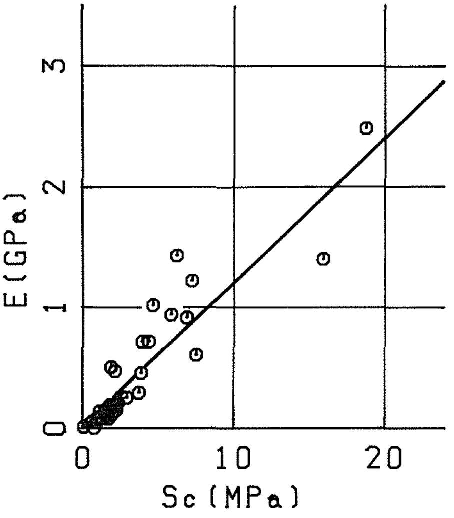

Fig.4 Relation between Young's modulus and uniaxial compressive strength

Eq.(4) agree well with the observed data.
By increasing the discharge rate, abrupt decrease in piezometric head is observed. It should be noted that the recovery of piezometric head can be observed by decreasing the discharge rate. Eq.(3) and Eq.(4) can then be used to determine the change in piezometric head with time at each well.
The subsidence contour map obtained by calculation is shown in Fig.6. And Fig.7 shows the observed contour map at the same time. In Fig.7, the total volume of pumped out water containing natural gas at each well is also shown. Around the area of high discharge, land subsidence is greater, reaching a max. of 116mm at point x in Fig.6. And, it should be noted that the outline of subsidence follows the distribution pattern of wells.

The calculated and observed subsidence contour maps are fairly similar.
For purposes of comparison, subsidence profiles of section a-a' and b-b' are shown in Fig.8.

The calculated subsidence (max.121mm) is a little larger than that observed (max.110mm). But the subsidence tendencies for both are very similar, thus, our analytical method could be very effective in estimating subsidence due to the extraction of water containing natural gas.

Conclusion

In this report, we have shown the mechanism of land subsidence and the analytical method for predicting subsidence in a natural gas field.

The decrease in piezometric head in gas seams, that is, the increase in effective pressure,due to the extraction of water containing natural gas, causes land subsidence. In order to prevent this subsidence, we must avoid the abrupt increase in discharge rate. By decreasing the discharge rate, we can expect a recovery of piezometric head in the

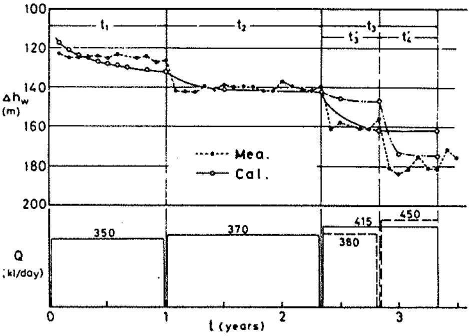

Fig.5 Variation in piezometric head with time

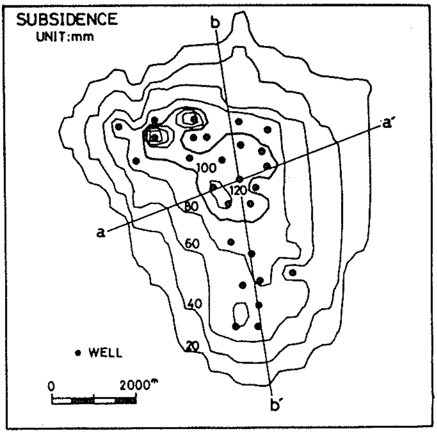

Fig.6 Calculated contour map of subsidence

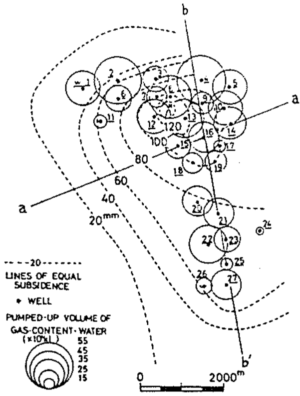

Fig.7 Measured contour map of subsidence

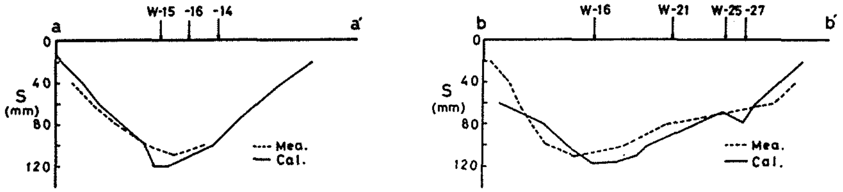

Fig.8 Subsidence profiles

gas seam, and probably some ground rebound.

Our analytical method enables obtention of a plan subsidence contour map. This is supported by agreement of our calculations with results obtained from in situ measurements.

Our method can thus be very profitably used to estimate the subsidence in natural gas fields, if properties of gas seams etc. are sufficiently investigated.

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SOIL UPHEAVING BY GROUTING TO SAFEGUARD ZONES AFFECTED BY SIGNIFICANT SUBSIDENCE PROBLEMS : ITS APPLICATION TO VENICE AS PECULIAR EXAMPLE

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ABSTRACT

The subsidence of Venice, at present essentially due to natural causes only, is characterized by very small rate. Nevertheless, the general situation of the lagoon town is still dramatic, because of the high subsidence values occurred in the past. The suggested safeguard treatments are essentially based on the regulation of the tidal levels of the lagoon area by means of mobile barriers system, meant to prevent formation of "exceptional" high waters.
Consequently this solution requires the complementary safeguard of the city from floods provoked by sea levels of "medium-small" range, by intervening in the more subsided areas by means of barriers and upheaval of each single island or of specific built-up zones. To this end, in 1971/72 an upheaving experiment on an area of Poveglia Island was successfully carried out by means of pressure grouting in the underlying ground.
In this paper the various aspects of the experimented system are analysed and discussed in view of a possible application within the foreseen solutions for the safety of Venice.

The actual rate of subsidence of Venice essentially due to natural causes is very small. Nevertheless, the general situation of the lagoon town is still dramatic because of the high subsidence values occurred in the past, induced mostly by anthropic factors such as to make unacceptable life conditions of the historic centre.
Floods occurred in recent years with an evergrowing frequence have made the life conditions of the historic centre unacceptable.

The suggested safeguard measures are essentially based on the regulation of the tidal levels of the lagoon area by means of a system of mobile barriers for preventing the formation of "exceptional" high waters.

The requirement of limiting the frequence of these regulation measures is dictated by the necessity of providing the lagoon with the required water exchange and guarantying the maritime navigation that is essential for the social and economic requirements of the region.

Consequently this solution requires the complementary safeguard of Venice from floods caused by sea levels of "medium-small" range, by intervening in the areas of pronounced subsidence by means of protection or upheaval of each single island or specific built-up zones.

These operations must be carefully studied case by case, and may consist in the superelevation of open spaces and execution of perimetrical protections along the canals.

However the logistic and structural conditions either of some areas or some monumental zones may require an "indirect" upheaval, by treating the underlying strata whenever it is not possibile to intervene directly on the structures themselves.

To this end, in 1971/72 an upheaval experiment on an area of Poveglia island, a few kilometers far from the historic centre, was successfully carried out by means of grouting in the underlying ground (fig. 1).
(The choice of Poveglia, a small island with several buildings, has been made with the support of the "Magistrato alle Acque" and of the Hydrographic Institute of Venice; the experiment has been carried out with the approval of the Competent Authorities and under the control of Government Officials).

The test area consisted of about nine hundred square metres including two small old buildings that were particularly interesting for the purpose of the test because of their extremely poor structural conditions.

The area was about 5 meters far from a large building and was separated from the sea by a 10 meters wide strip of land.

The main treatment parameters have been established on the basis of an accurate preliminary analysis of the structural conditions of the buildings and the geotechnical characteristics of the surrounding ground, consisting of a typical lagoon sedimentary soil with alternations of fine sands and silty-clay layers.

In particular, the nature of the soil has been investigated by means of exploratory borings, laboratory geotech-

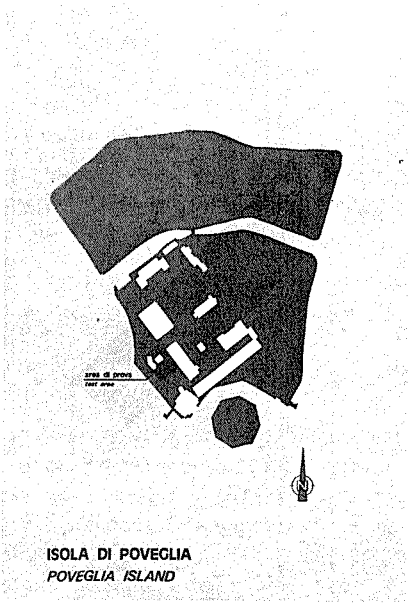


Figure 1

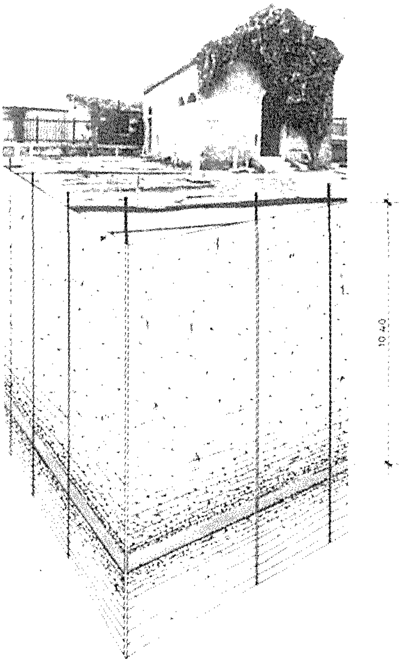


Figure 2

nical tests on several samples and static penetrometer tests.

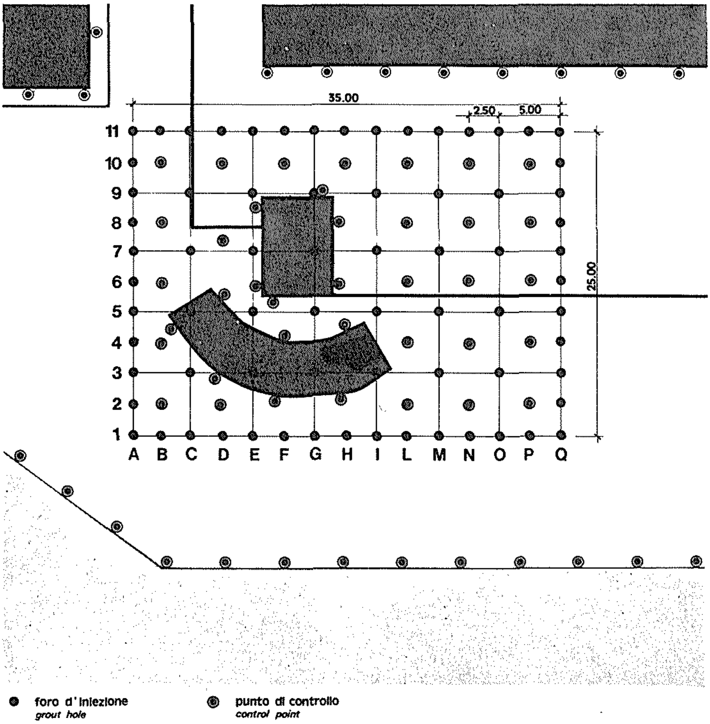
Soil upheaval by grouting consists essentially in injecting at pre-established depth a mix that causes a horizontal rupture of the soil with a resulting formation of a new layer and the consequent homogeneous lifting of the overlying ground (fig. 2).

The horizontal spread of the mix easily occurs in the sedimentary soils of the lagoon area where the ratio of horizontal to vertical permeability is very high.

The geometrical features of the Poveglia treatment, i.e. depth of grouting and spacing between the grouting points has been established so as to :

- limit the quantity of grout
- guarantee a great flexibility of treatment in order to ensure precision upheaval of specific areas.

Grouting depth was about ten meters from surface where a thin sandy layer was sandwiched between two layers of clay. The spacing of the grouting points was 5 meters, reduced to 2,5 meters along the perimeter in order to overcome the shear resistance of the soil (fig. 3).



UBICAZIONE FORI DI INIEZIONE E PUNTI DI CONTROLLO DEI SOLLEVAMENTI
LAY-OUT OF GROUT HOLES AND CONTROL POINTS OF UPHEAVAL

Figure 3

Grouting has been executed through a series of vertical sleeved pipes. This is a widely adopted technique for the treatment of soils, that makes it possible a repeated mix injection at various levels with pre-established rate of delivery and pressure.

The grout material has been defined after laboratory and field preliminary testing. It consisted of a cement-clay mix with cheap additives and characterized by non-polluting components, medium-high initial viscosity, high thixotropy and volume yield, and slow strength increase (final value of the order of 20 kg/cm^2).

The grouting plant consisted of two complementary sections : the one for the preparation of the clayey suspension, the other for mixing it with cement and additives, automatically controlling the grout by volume, and pumping it to the grout holes.

The system was controlled from a monitoring cabin where a specialist operator was continuously surveying quantities injected, grouting pressures and upheaval values (fig. 4).

The operative control of uplifts has been obtained by means of a specially designed system of electric cells located on the surface and connected to the building structures, inside and outside the test area (fig. 5).

The cells were interconnected by means of a constant level water circuit which in turn was part of an electric system. The rising of a cell was immediately monitored on an electric panel displaying the lay-out of the test zone.

The remarkable sensitivity of the system allowed the immediate and precise detection of localized upheavals, and the possible adjustments through adequate grouting operations.

This system, daily checked through precision levelling, proved extremely efficient in monitoring the treatment.

Outlined hereinafter are some details of the operations.

Four grout pumps have been employed for the simultaneous treatment of the four sectors in which the test zone had been subdivided.

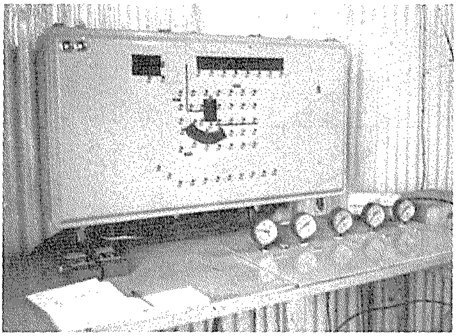


Figure 4

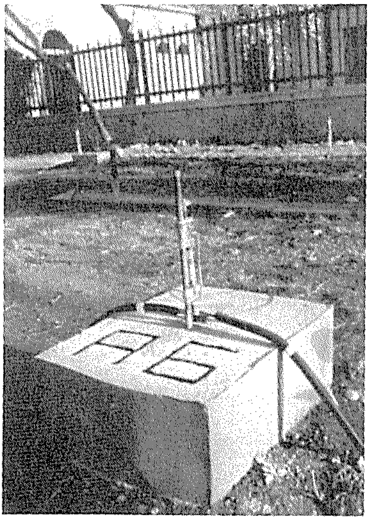
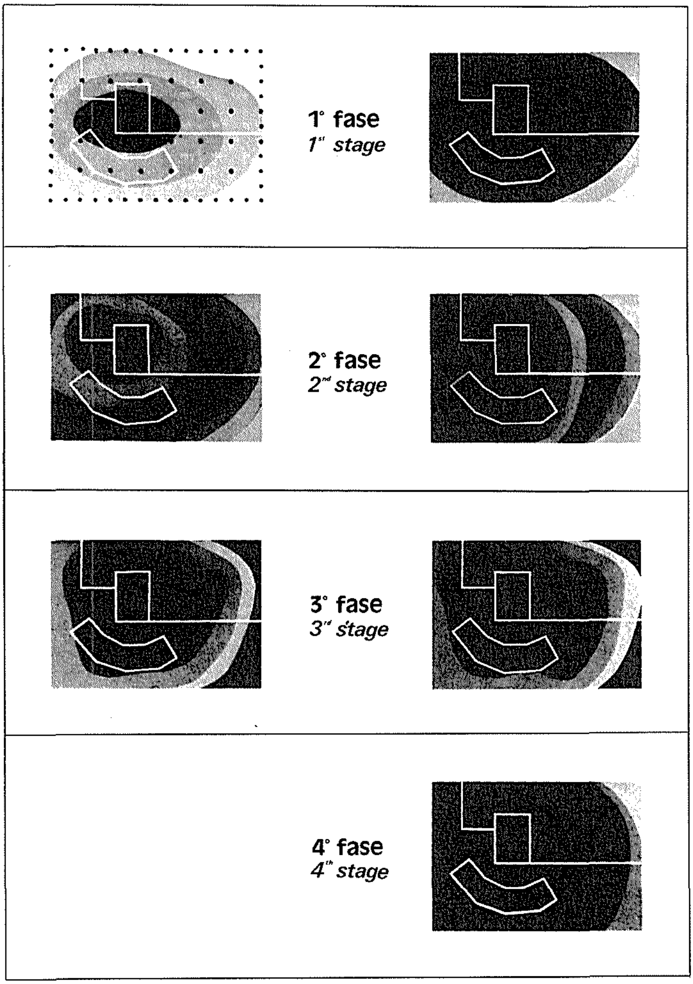


Figure 5

The first grouting has been carried out along the perimeter in order to overcome the shear resistance of the soil and limit the escape of grout outside the theoretical area.
Subsequently a systematic grouting of all the internal points has been executed by repeated injections of pre-established volumes of mix.
Once initial upheaval was achieved, localized groutings have been carried out in order to obtain a uniform rise of the ground (fig. 6).

The accuracy of the result obtained from the start of the work has been quite remarkable. As a matter of fact, it was possible to carry out a homogeneous lifting, by grouting selected points on the basis of the localized upheavals



ISOIPSE IN CORSO D'OPERA
CURVES OF EQUAL UPHEAVAL DURING THE COURSE OF WORK

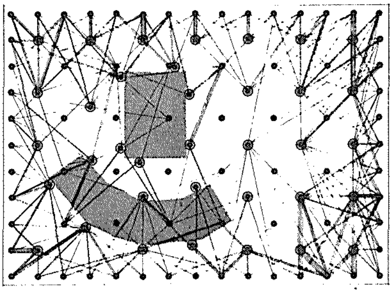
Figure 6

as shown by the monitoring system.

Based on this preliminary experimental results, it was decided to subdivide the treatment by lifting first the two small buildings and then the complementary area.

It must be noted that the ground does not react isotropically to the grouting, and it is necessary therefore to determine in course of work which specific grouting points cause the specific localized upheavals (fig. 7).

In practice, the sistematic recording of the correlation between points being injected and zones being lifted at the same time provided the statistic program of subsequent grouting operations.



● foro di iniezione
grout hole

◉ punto di controllo
control point

CONTEMPORANEITÀ TRA SOLLEVAMENTI LOCALIZZATI E FORI INIETTATI
CONTEMPORANEITY BETWEEN LOCAL UPHEAVALS AND HOLES BEING GROUTED

Figure 7

In view of possible larger scale treatments to be executed in more delicate and complex conditions, a computer program has been now elaborated. This program analises the results as they become available in course of work and defines the scheme of subsequent grouting. In practice this is a dynamic model which is continuously supplied with the latest results obtained and automatically feed the grouting plant with the updated injection scheme.

The pre-established upheaval of 10 centimetres has been achieved during 75 days by the low pressure injection of about 1000 cubic metres of mix.

Based on the design requirements, the magnitude of upheaval can be increased by employing proportional quantities of grouting, as clearly shown in the figure 8 where injected volumes and results achieved are compared. In fact after the initial phase of the treatment, the average absorption for unit of surface and upheaval remains fairly constant.

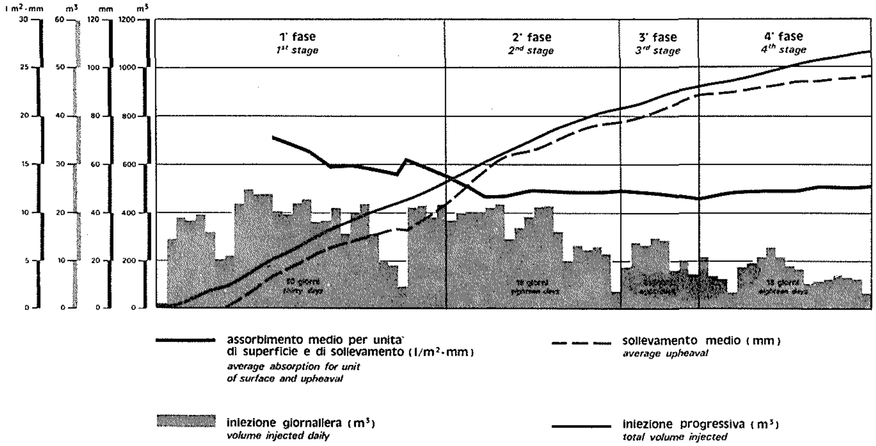
If the specific absorption per upheaval unit is considered, the notably bigger grout take along the perimeter can be observed (fig. 9). Consequently the specific quantities of work are deemed to decrease remarkably with the increasing of the area to be treated.

It is worthy mentioning that the differential ground upheavals recorded in course of work have been always lower than the deformation values congruent with the static characteristics of the buildings.
These values had been preliminarily verified through an accurate structural analysis.

The buildings in fact have never shown any sort of damage.

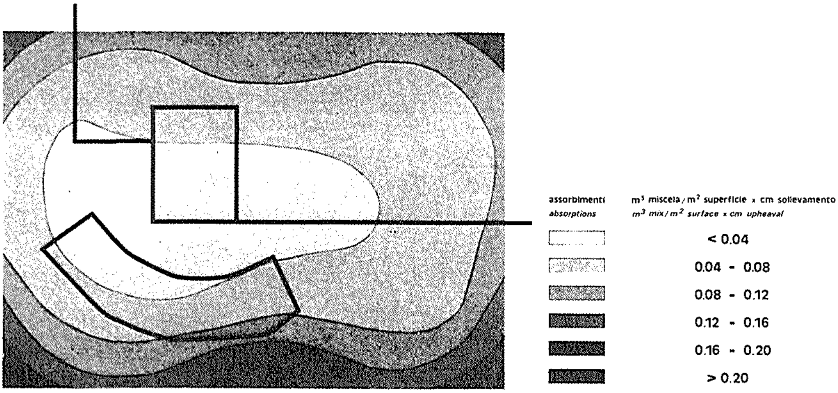
The gradient of the upheaval at a boundary of the theoretical area has been accurately checked during each operational phase.

As shown in figure 10, the upheaval is nil at 4.5 meters from the theoretical contour line towards the adjacent building, and at about 8 meters on the other side where special precautions had not been taken.



DIAGRAMMI DI ASSORBIMENTO DI MISCELA E SOLLEVAMENTO
MIX ABSORPTION AND UPHEAVAL DIAGRHAMS

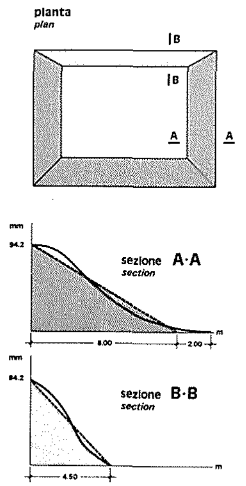
Figure 8



ASSORBIMENTI SPECIFICI PER UNITA' DI SOLLEVAMENTO
SPECIFIC ABSORPTION PER UPHEAVAL UNIT

Figure 9

fig. 10



GRADIENTE DEL SOLLEVAMENTO AL LIMITE DELL'AREA TEORICA

UPHEAVING GRADIENT AT THEORETICAL AREA BOUNDARY

Figure 10

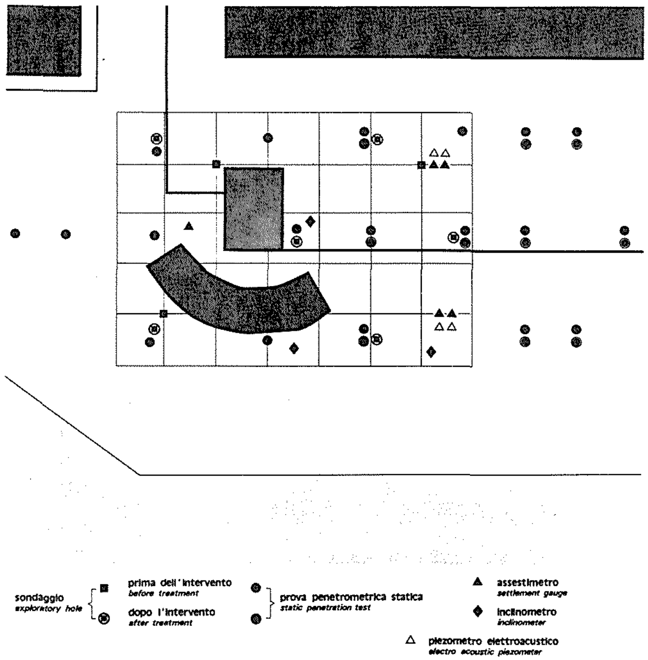
If required by particular boundary conditions, this gradient can be furtherly reduced by extending in time through successive injection phases the perimetral treatment.

During and at the completion of the treatment the soil characteristics have been accurately checked and recorded by means of settlement devices, inclinometers, piezometers and static penetrometer tests (fig. 11).

Exploratory borings executed after completion have statistically revealed depth, thickness and final characteristics of the layer of grouted mix.

Besides the levelling carried out systematically during a ten years period has proved the stability of the result achieved (fig. 12).

The experiment, which matched quite closely the design forecast, required par-



INDAGINI E STRUMENTAZIONI GEOTECNICHE

GEOTECHNICAL SITE INVESTIGATION AND INSTRUMENTATION

Figure 11

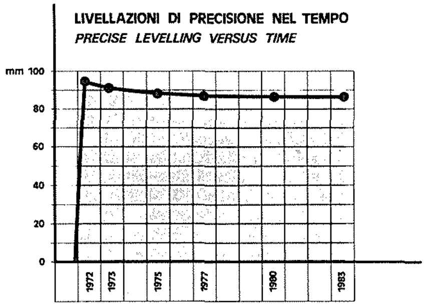


Figure 12

ticular ingenuity in its practical implementation and showed that :

- upheaval can be obtained according to the predetermined phases, rigorously respecting the specified differences of level among the various points
- *it is possible to control the uplifting* operation since the effects can be limited to the testing area without affecting the neighbouring areas
- the quantities of work and materials and time of execution are within the magnitude admitted for the restauration of valuable urban areas and historic towns.

In other words the validity of the system proposed was clearly evidenced, and this from both a strict technical point of view and the function that the system itself may have in helping to solve the problem of Venice.

A further testing phase will have to be undertaken to get a model design in view of a larger scale application which taking into account the diversified structural situations prevailing in Venice.

In any case it has been already proved that the method proposed offers the following features :

- flexibility and selectivity
- feasibility, and this according to a set of priorities and schedule that can be perfected *as the work progresses*
- wide possibility of repeated upheaval grouting in the same area.

EVALUATION AND PREDICTION OF SUBSIDENCE IN OLD WORKING AREAS AND PRACTICAL PREVENTIVE MEASURES AGAINST MINING DAMAGE TO NEW STRUCTURES

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Abstract

The shortage of adequate construction sites has compelled many engineering structures to be planned in old mining areas in Japan. This paper shows, first, the characteristic of the excavated underground environment and, next, the necessary measures for the safety of surface structures. Where the depth of a mined out area is relatively shallow, there is a tendency of cave-in consequent to surface loadings. The effect of surface loading is inversely proportional to the depth of working. In the case of heavy structures with limited allowable deformation, special preventive measures against differential subsidence must be taken into consideration. Finally, a power plant project is studied as an example.

Introduction

Subsidence due to mining has caused various kinds of damage to the surface structures, and actually great deal of mining damage becomes the severe social problem at some time in many coal fields, but the most of the mining damage has been restored at present.

Recently, the effect of old coal working on the deformation and the bearing capacity of their base grounds of newly constructed structures has become into question, because it was obliged to plan many engineering structures in old mining areas due to the shortage of adequate construction sites and the difficulty of getting location. That is, buildings, dams, bridges for railroad and expressway, power plants, tunnels, storage tanks and so, have to be built on the ground which has old coal working.

Under these circumstances, the ground with the old coal working is affected by the weight of structures and may have further subsidence, cave-in and the decrease of bearing capacity. This behavior of the ground may make the damage to the structures.

These phenomena are frequently proceeded by fluctuation of the ground water level and earthquake occurrence. In these cases, it is very necessary and important to estimate the ground movement (subsidence and cave-in), in advance, for the design of structures or for the judging feasibility of construction.

The authors have studied these problems fundamentally and examined about 100 practical cases.

As a result of them, we must consider that the phenomena are clearly different with the mining depth. Firstly, this paper shows the effect of old working at shallow depth, and preventive measures against the ground movement for construction of the surface structures. Next, we show some practical cases which have old coal working in relatively deep depth.

The Effect of old Working at shallow Depth

Characteristics of Cave-in.- The ground with old coal working at shallow depth has often many unfilled cavities. The strata above these cavities mainly consist of soft rock such as weathered or fractured sandstone and shale. Due to the load of structures, the change of ground water level or earthquake occurrence, the strata over cavities may deteriorate and collapse, or the pillar may be crushed with time and cavities may eventually migrate to the surface and cause sudden cave-in. Photo 1 shows the typical example of cave-in due to the drainage of ground water for the construction work of new railroad Shin-Kansen. Fig. 1 shows the monthly frequency distribution of cave-ins occurrence in West Japan district. In the district, there were so many coal mines for about 100 years , and many cavities remained at shallow depth, and we have 50-60 cave-ins every year. Cave-ins tend to occur during rainy season

Photo 1 The typical example of cave-in due to the drainage of ground water

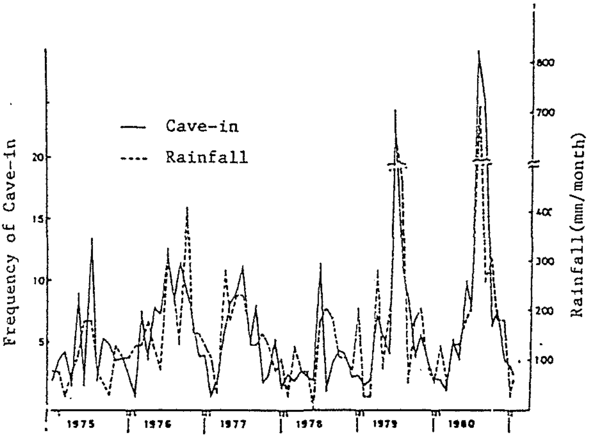

Fig.1 The monthly frequency distribution of cave-in

(from May to July)or at heavy rainfall. We have observed the direct relationship between cave-in and rainfall. It is the reason for the increase of effective pressure in the ground. Fig. 2 shows the location where cave-in have occurred in a part of the district from 1975 to 1980 as an example. It is noted that they have occurred near along the outcrops and at the depth within about 30 m . These cave-ins came out not only at the time of working but also after the long time from working. The dimension of cave-in is about 3 m in diameter and its depth of falling is about 2 m, generally. The side wall of cave-in commonly shows the inverse funnel shape.

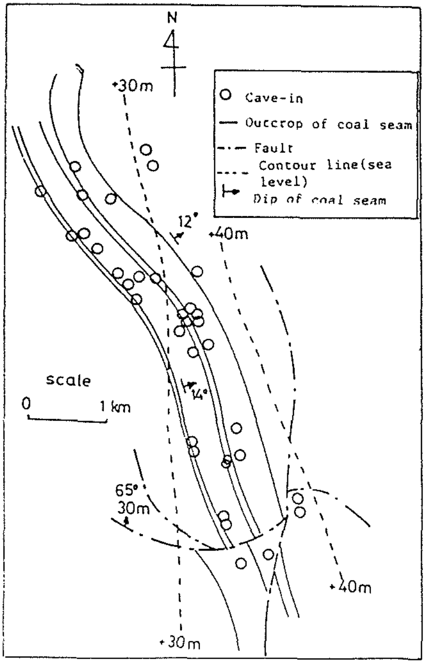

Fig. 2 The location of cave-in near the outcrops of coal seam

To illustrate the earthquake effect on the cave-in occurrence, we shall take the example of the Tohoku district in North-East Japan where lignite has been mined mainly at Kitakami, Miyagi, Mogami and Souma coal fields since about 1890. On June 12. 1978, an earthquake of 7.4 magnitude, named as Miyagi-Ken-Oki earthquake, occurred off the coast of Miyagi-pref. Fig. 3 shows the main lignite coal fields in the Tohoku district and the seismic intensity distribution. According to the investigation, about 15-20 cave-ins normally occurred every year, but in 1978, 219 cave-ins came out and this unusual increase was attributed to the earthquake. Especially, at Ohhira in Miyagi coal field, the maximum vertical acceleration was about 100 gal and 22 cave-ins occurred in a

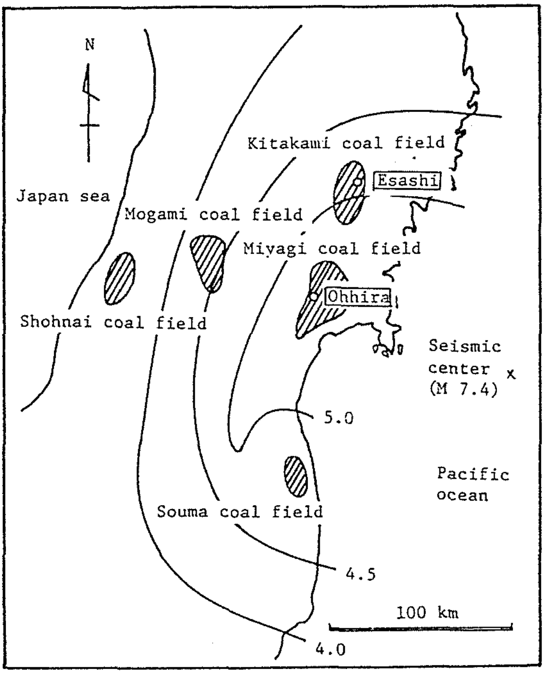

Fig. 3 Distribution of the main lignite coal fields and seismic intensity

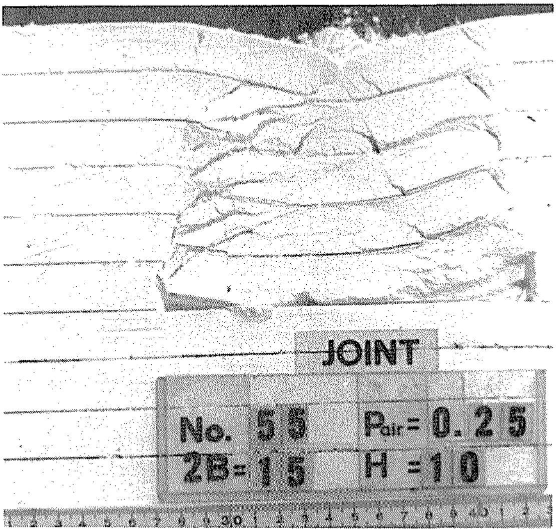

Photo 2 The typical view of cave-in by the base friction apparatus

5000m² farmland. In this district, many mines were worked at very shallow depth by the pillar and stall mining method. The lignite seam, 2.0 - 3.0 m thick, was widely distributed 2.0 - 3.0 m below the ground surface.

The other side, at Esashi in the Kitakami coal field, cave-in occurred at the depth of about 30 m below the ground surface, even though the maximum vertical acceleration was about 50 gal.

The incremental load procedure of the finite element method is an useful means in determining whether cave-in will occur or not, theoretically. Whether cave-in occurs or not, is determined by the developement of plastic zone and its geometrical shape. But, numerical calculation to simulate realistically complex geotechnical situation such as discontinuity of ground is still unsatisfied. In this cases, it is convenient to perform behavioural experimental studies by physical model test. We have tried many kinds of physical model test such as agar-agar model, and model by downward movement of the base and centrifuge model for the representation of gravity in model. The base friction technique is very well suitable for near the surface problems such as cave-in. Photo 2 shows the typical view of cave-in experiment by the Egger's base friction apparatus with air pressure.

<u>Preventive Measures for Cave-in.</u>- When heavy structures are built on the ground which has cavities at shallow depth, we must consider the preventive measures against cave-in. The suitable preventive measure must be adopted with the following conditions; mechanical properties of ground, depth and width of cavity, mining style, loading, cost and the allowable deformation of surface structure.

There are 4 types of preventive measures which we have examined.

1)Grouting; it will be undertaken only where the cavity is in small scale because grouting is very expensive.

2)Excavation and filling; it is used when the cavity is comparatively at shallow depth and this method is more economical than grouting.

3)Pilling until below the floor of the cavities; this method can be used for soft ground but may not be used for superposed cavities. In this case, we must take the negative friction on piles into consideration.

4)Foundation in the form of slab; the foundation to resist cave-in is expensive, but it is very useful in the case of small cavity.

Old Working at Deeper Depth

General.- The effect of surface loading is inversely proportional to the depth of old working. Differential subsidence could cause very serious problems. The causes of differential subsidence could be attributed to the existence of cavity with disturbance around the region of its roof and/or pillar contraction. In gereral, planned structures should be designed to minimize their dimension and own weights as much as possible and to reduce the friction between the ground and their foundations in order to reduce the stress in the structure.

But, in the case of heavy structure with very limited allowable deformation, the ground movement caused by loading as well as the movement resulting from mining engenders serious problems and must be estimated in advance. The movement should be maintained within the allowable value by means of inserting deep slab or using jacking system to restore it to a level attitude. If the ground movement causes little or no appreciable structural damage, it is assumed to be within the allowable value. This allowable value as well as that due to mining is relative and subjective hence it depends on the kind of sturcture and on human sensibility.

Study of a Power Plant Project.- A study of subsidence with respect to a power plant project is given as a practical example. The site was excavated till before 20 years at a depth of 20 - 150 m by the room and pillar, and short wall methods. This new power plant project is now being materialized.

The layout of the plant is such located that it avoids the fault and the shallow mining areas as shown in Fig. 4.

Ground survey and analysis.- Although the phenomenon of ground movement has to be considered as being three dimensional, for practical purpose, we shall consider about 4 vertical sections as plane strain condition. Each of the section is divided into meshs for finite element analysis and consists of 9 band layers. These section are taken based on the finding from geological surveying. From the mining record , it is found that widths of room and pillar are 60 m and 10 m respectively and height of excavation is 1 m. We adopt the velocity of propagation Vps by sonic logging as the index for dividing band layers. Because it was found the values of Vps mostly agreed with the change of the plant and the mechanical properties of ground in each depth compared with other test results.

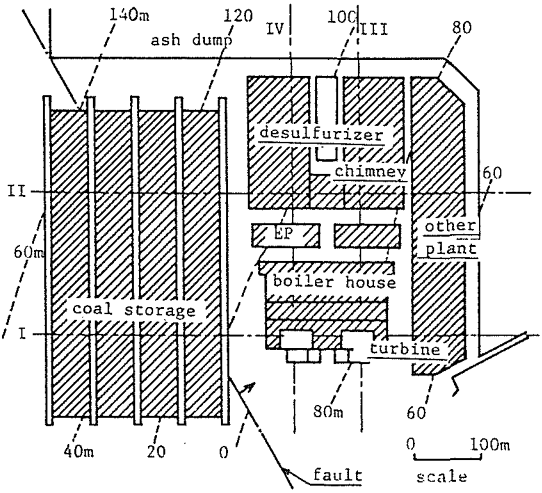

Fig. 4 The layout of the plant and the contour line of old working area

Finite element analysis is

performed by following the steps of construction as shown in Table 1. The mechanical properties of rock mass are tested by core triaxial test, in situ dilatometer test and geophysical logging test. The deformation coefficient E is decided from the mean value of Eps (derived Vps through Dixson's examination with risk ratio 0.2). The rock mass strength is derived from the value of E and the relation of shear strength in brittle material. These values are corrected by the trial analysis of mining in order to decide the optimum input data (Table 2). That is, we regard the mechanical properties of rock as the input data being satisfied by the following conditions;

(1)the plastic zone due to mining agrees with it from borings (see Fig. 5).

(2)The surface subsidence and the convergence of cavity due to coal mining can be appropriately compared with the precalculated value (see Fig. 6).

Table 1 Change of plant loads with each steps of construction

Composition	Step 3.* Excavation of Base	Step 4. Foundation & Structure	Step 5. Machinery
Turbine Foun.	-20.4	+15.5	+14.9
Turbine Bld.	- 4.9	+10.6	+ 4.6
Boiler Bld.	- 5.8	+ 5.8	+12.0
Chimney	- 3.4	+ 5.0	+10.0
Desulfurizer	- 3.4	+ 5.0	+10.0
EP & Others	- 3.4	+ 5.0	+ 5.0
Stock Yard	-	-	+ 8.0

(Unit: t/m^2)

* Step 1,2 and 6 is Mining, Cut and Embankment, and Seismic Load, respectively.

Table 2 Mechanical properties

Layer	Unit Weight $\gamma(t/m^3)$	Poisson Ratio ν	Fric. Angle $\phi(°)$	Cohe. C (t/m^2)	Defor. Coeff. $E(t/m^2)$	Velocity of propagation vps(km/sec)
I	2.43	0.34	40	33.0	41,400	2.51
II	2.32	0.39	40	74.0	94,100	2.83
III	2.38	0.34	40	56.0	71,300	2.66
IV	2.41	0.34	40	53.0	67,500	2.97
V	2.42	0.32	40	63.0	79,100	3.14
VI	2.53	0.30	40	131.0	165,000	3.35
VII	2.51	0.28	40	68.0	81,800	2.93
VIII	2.48	0.27	40	54.0	69,300	3.16
IX	2.49	0.35	40	53.0	61.300	2.69

Table 3 Differential subsidence

Location	Loading Condition	Max.Tilt $(x10^{-3})$	Min.Radious of Curvature (m)	Max. Strain $(x10^{-3})$
turbine basement	normal seismic	0.22 0.60	83,000 40,000	- -
ground surface	normal seismic	0.90 1.18	14,000 8,300	0.12 0.56

Result of Analysis.- Fig. 7 shows the subsidence profiles in section I under total load of structure. The maximum subsidence by non-linear and linear analyses is 47 mm and 34 mm, respectively. The differnce of them is caused by the effect of the plastic zone. If the ground has no mining, the subsidence may decrease about 30% compared within the case of mining. The trough leaning to the left may be the efect of the fault.

Fig. 8 shows the increment of subsidence with each step of construction. Step 4 is for the structure load (foundations and buildings) and step 5 is for the machinery load. The deformation in step 6 is the response at Kv=0.07, Kh=0.14 by the static seismic design method. Similar calculations are made in other sections. As a result of analysis, the maximum values of differential subsidence are confirmed within the above-mentioned allowable value by adopting the deep slab foundation as shown in Table 3. We intend to measure the subsidence during the process of the actual construction in order to compare with these analytical results in the future.

Conclusion

The roles of ground mechanical properties, earthquake, width and depth of underground cavity, existence of water and surface loading on the mechanism of cave-in have been clarified from our practical and theoretical studies.

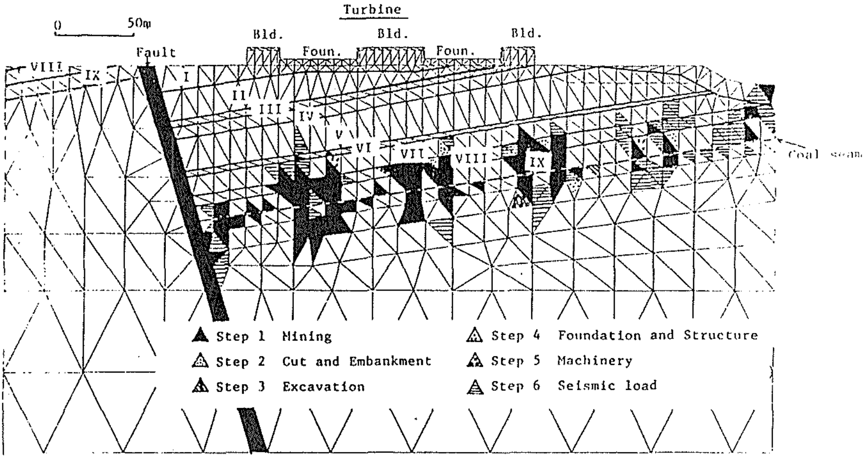

Fig. 5 The plastic zone due to mining and loading (Section I)

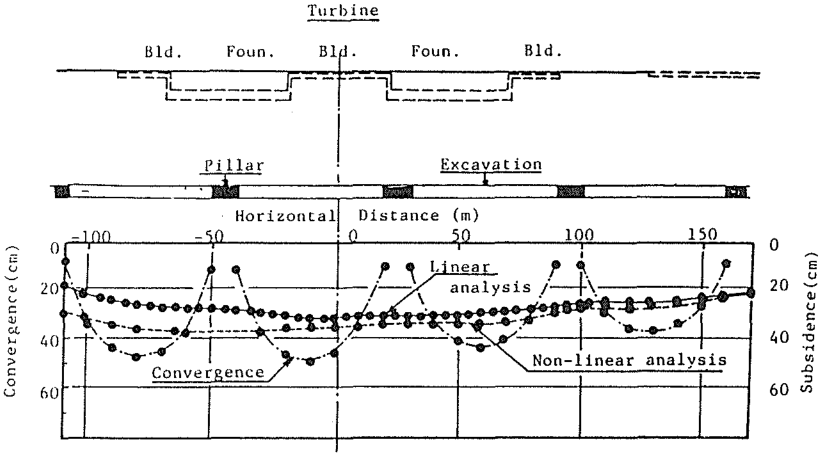

Fig. 6 The surface subsience and the convergence of cavity due to mining (Section I)

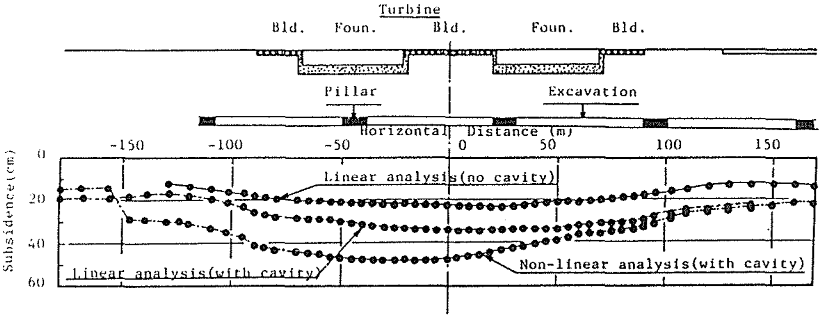

Fig. 7 The subsidence profiles under total load of the structure (Section I)

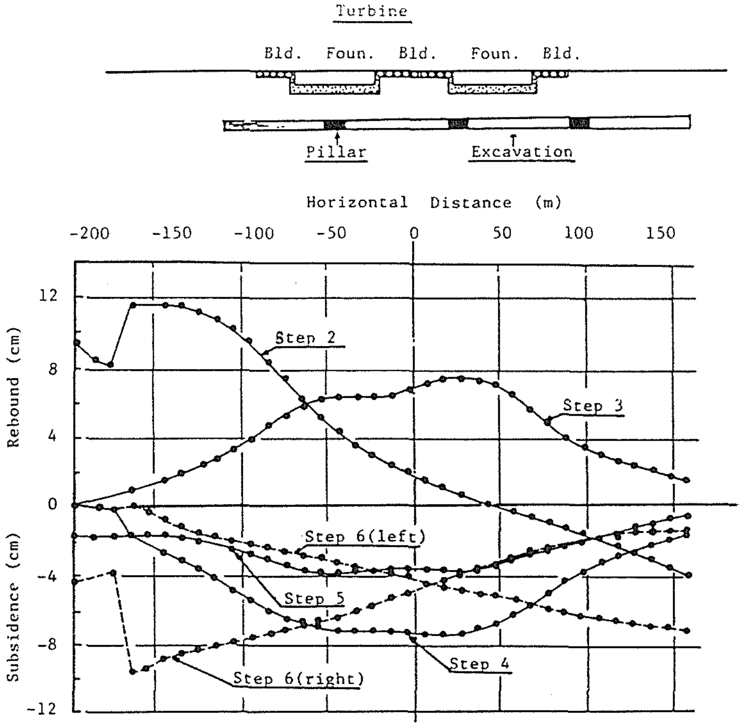

Fig. 8 The increment of subsidence with each step of construction (Section I)

Theoretically, the incremental load procedure of the finite element method is a useful means in determining whether cave-in and differential subsidence will occur or not.

Experimentally, the base friction apparatus with air pressure is very suitable for the study of problems near the surface, especially in relation to grounds with discontinuities.

We have made fundamental studies on the evaluation of subsidence in old working areas and have suggested practical preventive measures against damages to planned structures. Our suggested preventive measures have led to practical solutions in many cases.

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"SUBSIDENCE OF BANGKOK CLAY DUE TO DEEP WELL PUMPING AND ITS CONTROL THROUGH ARTIFICIAL RECHARGE"

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Abstract

Bangkok is located on a flat deltaic-marine plain 0.5 to 1.5 m above mean sea level. In the past two decades increased pumping of groundwater within the metropolitan area has led to the reduction of pore pressures, compression of soils and surficial deposits, drastic lowering of piezometric levels and ground settlements of more than 0.5 m with a maximum annual rate of 10 cm in some parts of the city. In this paper, the possibility of restoring the piezometric heads in a multiacquifer system through artificial recharge is explored.

Introduction

Bangkok, the capital of Thailand is situated on the Chao Phraya river 28 Km North of the Gulf of Thailand. The present population of the city is 5 million and it was once known as "The Venice of the East". Bangkok is laced by dozens of large canals (Klongs) which drain to the Chao Phraya River. The water table is at or near the surface throughout the city. Consequently, heavy monsoon rains quickly flood the major parts of the urban area during rainy season.

The City is presently sinking at an average rate of 5-10 cm per year and in 23 years has subsided a maximum of 120 cm in some areas. The problem has become so severe now as parts of the city are already below sea level. Land subsidence has created a bowl-shaped depression in the south-eastern metropolitan area and flood water resides there for much longer periods and drainage has become less efficient due to the reduced gradients in canals and storm drains.

More than 30% of the water consumption in the metropolitan area is extracted from acquifers beneath the city. In 1982, ground water use exceeded 1.35 million m^3 per day from more than 11000 wells. In the main acquifers, pumping has resulted in a lowering of the piezometric level from its original position near the surface to more than 50 m below the ground. Wide spread land subsidence is taking place throughout the metropolitan area east of the Chao Phraya River because of this excessive groundwater exploitation.

The first studies of land subsidence in Bangkok were undertaken in late 1960s and 70s (Cox, 1968; Paveenchana, 1970; Brand and Paveenchana, 1971; Brand and Arbhabhirama, 1973; Brand, 1974; Brand and Balasubramaniam, 1976). An extensive study involving field instrumentation at 31 stations has been carried out by the Division of Geotechnical & Transportation Engineering of the Asian Institute of Technology (AIT, 1981 & 1982, Nutalaya and Premchitt, 1981). The objectives of the study were to determine the extent and rate of settling, the effect of pumping on the magnitude and rate of sinking and preventive measures against subsidence.

Geomorphology and Geology

Bangkok occupies an area of about 1540 Km^2 near the southern margin of a low-lying, flat, marine plain termed the lower Central Plain (Fig. 1). The Upper

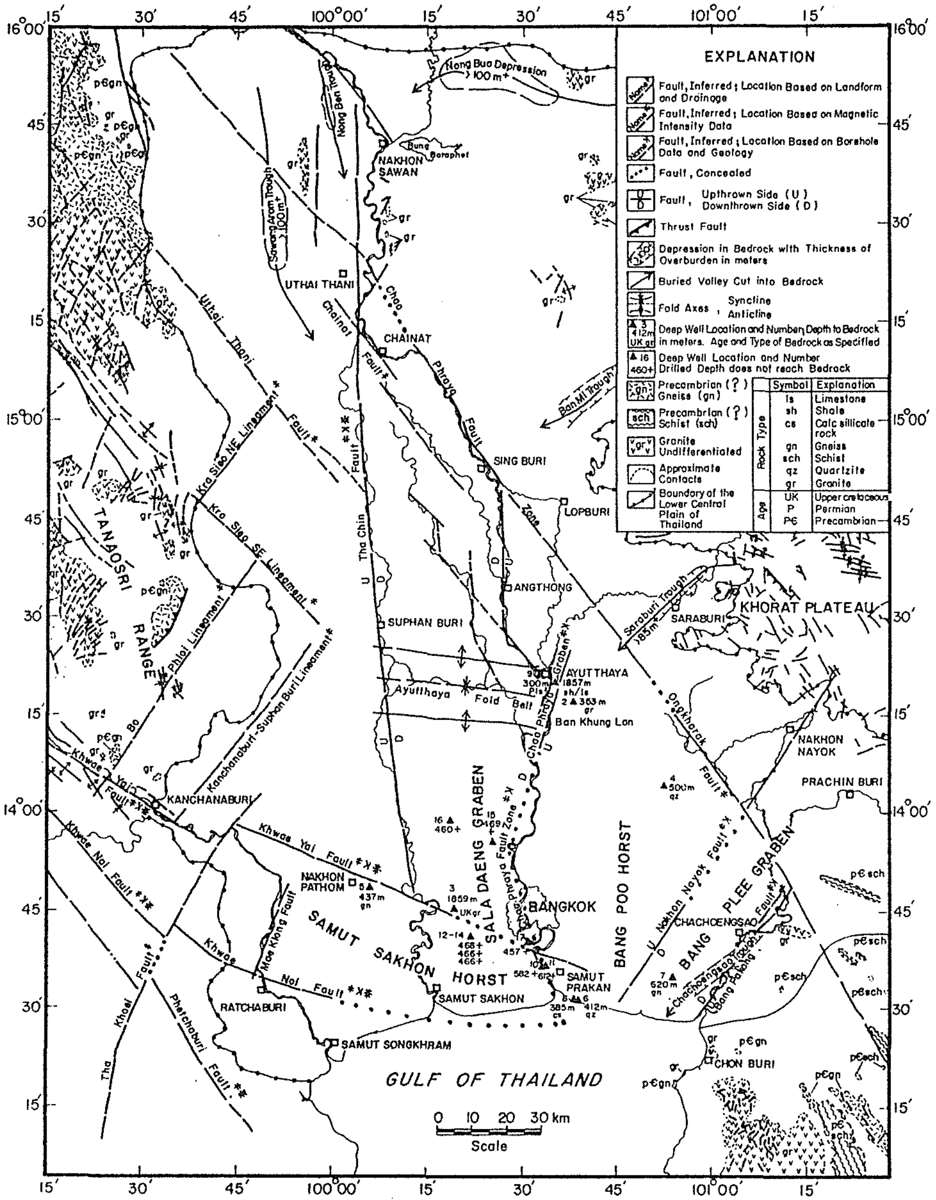


Fig. 1 Structural Framework of the Lower Central Plain of Thailand.

Central Plain begins where four rivers, the Ping, Wang, Yom and Nan, rising in northern Thailand combine to form the Chao Phraya River at Nakhon Sawan 240 Km north of Bangkok. The modern Chao Phraya has developed a meander belt about 10 Km wide down the centre of the Lower Central Plain. Its leeves are low and poorly defined and are not high enough to prevent widespread flooding of areas north of Bangkok, despite attempts made to control the monsoon flows.

The Central Plain and the Gulf of Thailand are located within a north-south trending structural depression which was generated by fault block tectonics during the Tertiary time. To the west the depression is bounded by the north-south trending Paleozoic fold belts of the Thai-Malay Peninsula. The eastern boundary of the basin is rimmed by the Khorat Plateau. Khrok Phra Arch at Nakhon Sawan borders the north, and in the south the depression extends southward into the South China Sea. Aeromagnetic data indicated that the Lower Central Plain is floored by basement arches and plutons in association with a diverse assemblage of faults or flexure zones (ACHALABHUTI, 1974). The exact configuration of the basin floor is unknown. A few wells which were drilled down to bedrock revealed several basement rock types at various depths from 1800 m to 350 m in the central part of the plain.

Aeromagnetic and seismic data covering the Gulf area also indicate an irregular basement floored by granite ridges and metasedimentary fold belts of the peninsular trend (KELLEY & RIEB, 1971). On land the aeromagnetic data indicate about 3,300 m of sediments on the coast south of Bangkok. A metamorphic ridge across the narrow part of the Gulf appears to close off partially or completely this basin from the main basin located in the south (CCOP-IOC, 1974).

Subsoil Characteristics

Subsoil profiles for engineering purposes have previously been established by various bodies for specific subsurface design and construction activities in Bangkok. The first attempt to delineate the subsurface strata in Bangkok in a systematic manner seems to have been made by MUKTABHANT et al (1966), in which three profiles within Bangkok city have been established. Expansion of construction activities in Bangkok during 1970-1980 brought further soil investigation over many areas which contributed to better understanding of subsurface conditions in Bangkok. The major projects that have conducted detail soil investigation over long sections (more than 5 km) are such as Bangkok Thonbury Ring Road, Water Transmission Tunnel System and Expressways. These recent results were summarized by POOPATH et al (1978) and typical profiles have been drawn over the Bangkok area. These profiles were restricted to the upper 30 m zone since most of boreholes were drilled upto 30 m only.

As a part of the investigation, two generalized soil profiles cutting through Bangkok area and extending over a great distance were established (Piyasena, 1982). The sections used to establish the profile are shown in Fig. 2. The north-south profile extends from the AIT campus to Pom Phrachul, a distant of about 60 km. The east-west profile covers a distance of about 45 km. from Nong Ngoo Hao in the east to Nong Khaem in the west. These two sections cut across the Bangkok area and cover the vital areas of interest in this investigation.

It can be seen that the soft Bangkok clay has an average thickness of about 14 m beneath the central Bangkok. The thickness increases towards the sea and decreases rapidly with distance to the north of Bangkok. Beneath the soft clay is a stratum of stiff clay which is generally about 5 m thick in central Bangkok, while the thickness decreases gradually with distance to the north and west of central Bangkok.

In the research study carried out at A.I.T. (A.I.T. Research Report No. 91) it was found that among many sand layers, the hydraulic and mechanical pro-

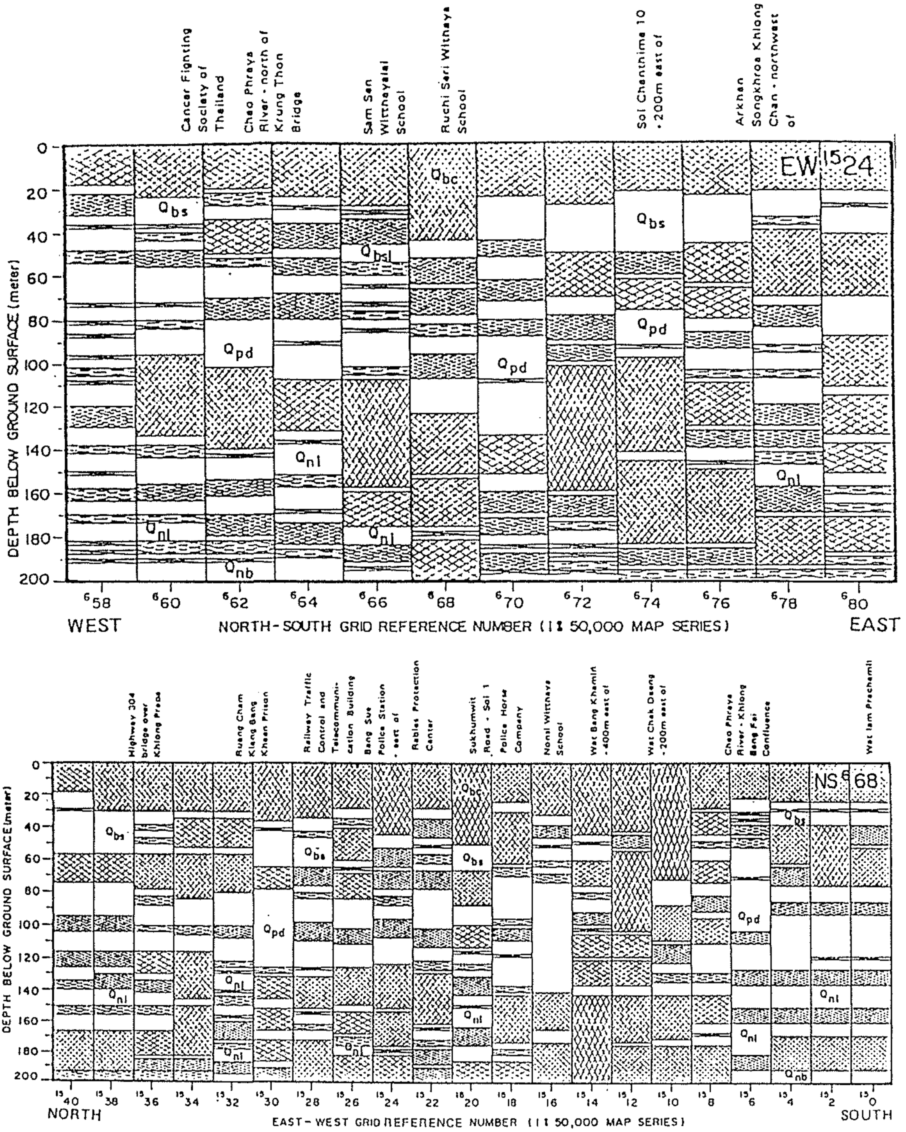


Fig. 2 Profiles of Subsurface Stratification

perties are approximately similar and the same is also true for the properties of different clay layers. To evaluate the hydraulic and compression characteristics of the whole subsurface body, it is not necessary to determine the exact depths and thicknesses of these strata, only the relative abundance of each type within a specified dpeth interval has to be known. The relative abundance of sand layers in 50 m intervals to 200 m depth have been established over the entire basin in this investigation.

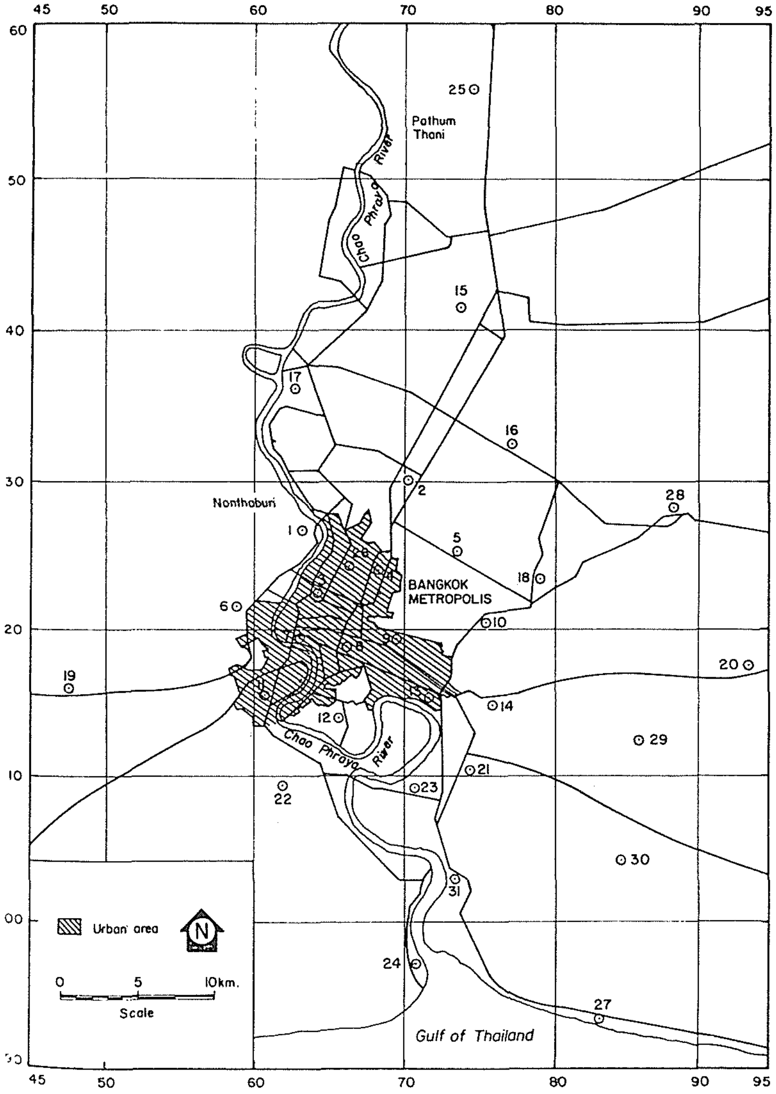


Fig. 3 Location of Subsidence Observation Station

Field Measurement

The observation of subsidence in Bangkok was carried out by installing field instruments at 31 locations as shown in Fig. 3. The monitoring of (i) piezometric levels at various depths and (ii) compression of various soil layers was started as soon as the instruments had been installed. Installation for the upper 50 m range at the first 24 station was completed in 1978 and the addition of deep instruments (upto 400 m) at seven stations was finished in 1982. For the measurements by deep compression indicators it

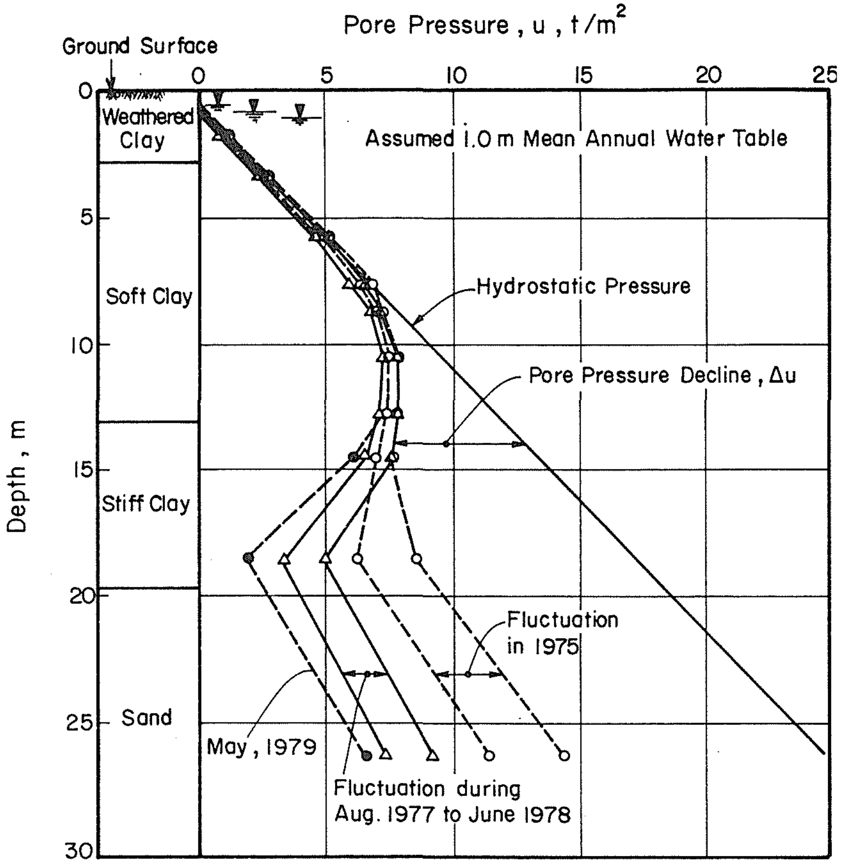


Fig. 4 Typical Variation of Pore Pressure in the Upper Clay Layer in Bangkok (Station No. 8).

was found that the results are almost equal to the surface subsidence obtained by the levelling survey conducted by the RTSD and the measurements can be regarded as absolute subsidence measurements. From the data gathered over the four-year period, the contribution of sub-soil compression to the total surface subsidence can be summarized as follows:

(i) Top 50 m depth range (top clay layer) - 40%, and
(ii) Zone deeper than 50 m - 60%

Figure 4 shows the variation of pore water pressure with depth at different time periods as observed from field piezometers at station 8, Chulalongkorn University. At this particular station the observatio has been conducted by AIT researchers since 1975 (NANEGRUNGSUNK, 1976; THAMMAKUMPEE, 1978). From this long-time observation, it is clearly seen that the pore water pressure in the top 10 m depth zone of the subsurface is within a narrow range and there are no appreciable changes in the pressure over the five year period. On the other hand, the water pressure in the zone deeper than 10 m dropped continuously over the period, with the maximum drop from

1975 to 1979 of about 6 t/m^2 (piezometric level drop of 6 m) at 20 m depth. There exists significant fluctuation of the water pressure with time during a one year period but the general trend is a net annual drop of pore pressure of more than 1 t/m^2. It may be concluded that the pressure in the top 10 m zone is stable and is only subject to small seasonal fluctuations over a one year period without significant influence from groundwater pumping. The water pressure in the deeper zone, however, seems to be influenced by the decline due to groundwater pumping as well as the seasonal fluctuation. The combination of two effects results in the variation of pressure with the time with a net drop in pressure increasing in the zone below 10 m.

Apart from the measurements carried out in this research, other independent measurements were also conducted by the two companion projects. For the project on "Groundwater Resources in the Bangkok Area: Development and Management Study" a net work of 60 observation wells over the Bangkok area were installed and monitoring has been conducted since completion of the installation and the results were obtained for a period of over four years at some of these observation wells. The project on "Surface Levelling of the Bangkok Area for the Determination of Land Subsidence" has carried out first order levelling over the Bangkok area with the reference elevation originating at a mountain range in Ratburi province. Seven runs of levelling have been conducted at about half-a-year intervals. The repeated levelling runs allow the estimation of the surface subsidence rate over the Bangkok area. These independent measurements were found to confirm the findings in the present investigation.

Attempt has also been made to reconstruct the contour of subsidence rate (Fig. 5) by using the results of last two levellings (1981-1982) and from the automatic subsidence recorders results. Again three zones of subsidence are distinguished. They are (1) above 10 cm/year, (2) between 10-5 cm/year (3) below 5 cm/year. The maximum subsided areas are concentrated in the eastern part of Bangkok, especially in the Phra Khanong and Bangkapi districts.

Analysis and Model Simulation of Subsidence

A mathematical model for land subsidence invariably consists of a hydrologic model, simulating the large aquifer basin with many pumping wells, and a consolidation of compression model, simulating the consolidation of the various sub-soil strata at specified locations. The declines of piezometric levels in the various layers of the hydrologic model are used in the consolidation model as the boundary piezometric heads for the clay layers to estimate compression and surface subsidence at every time step.

In this study, the mathematical model consists of a 'three-dimensional' hydrologic model connected to a one-dimensional consolidation model. A schematic diagram showing the relationship between hydrologic model and consolidation model is given in Fig. 6.

The model was devised to study a time-dependent problem. Normally, model simulation for a system is carried out to estimate and predict the future performance of the system. The parameters employed in the simulation were obtained from small scale field tests or laboratory tests on samples.

(a) Hydrologic model - For the mathematical simulation of a real multi-aquifer system, the model should take into account the inhomogeneity and anisotropy of the material properties. In the general case, the governing equation for the piezometric head in an aquifer in a multi-aquifer system can be expressed as:

$$\frac{\partial}{\partial x} T_x \frac{\partial h_i}{\partial y} + \frac{\partial}{\partial y} T_y \frac{\partial h_i}{\partial y} = S_i \frac{\partial h_i}{\partial t} + Q_i + W_i \qquad \ldots\ldots (1)$$

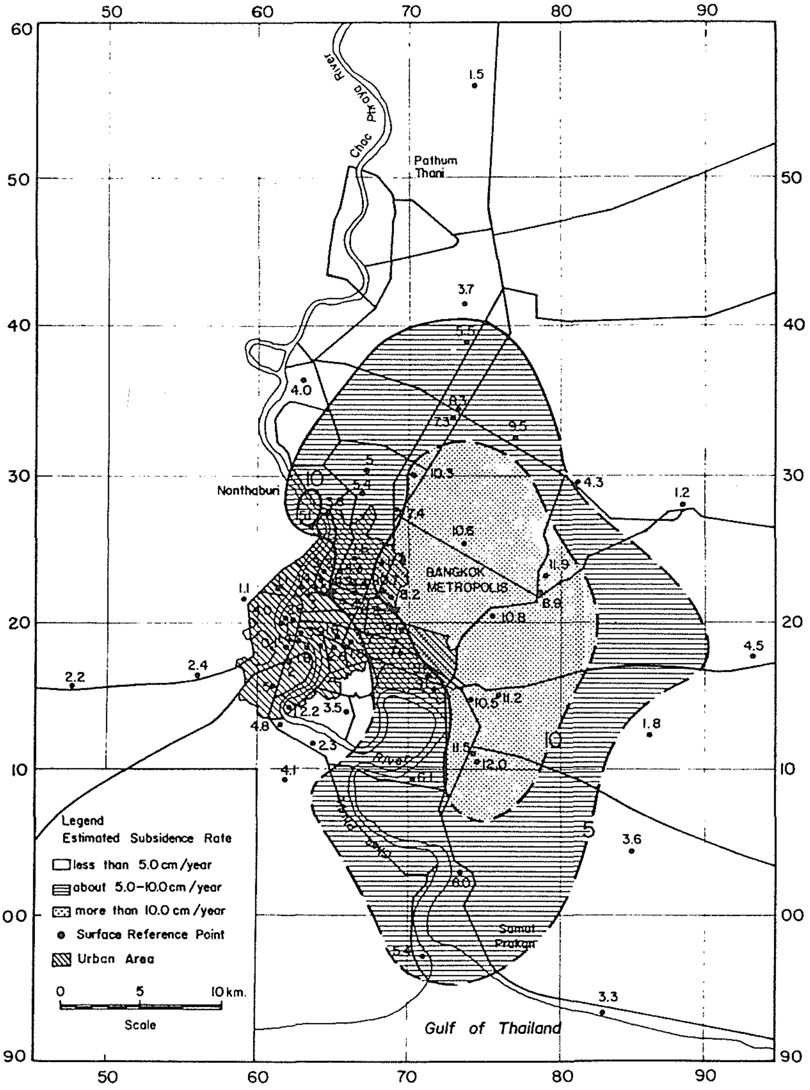


Fig. 5 Subsidence rate in Bangkok (cm/year)

where, i is the number assigned to the aquifer under consideration

h_i is the piezometric head in the aquifer

T_x, T_y denotes directional transmissibility of the aquifer

S_i denotes the storage coefficient

Q_i is the rate of ground water extraction out of the domain

The term W_i corresponds to the rate of ground water flow to the upper and lower aquifer in the domain, which includes (i) the direct seepage between

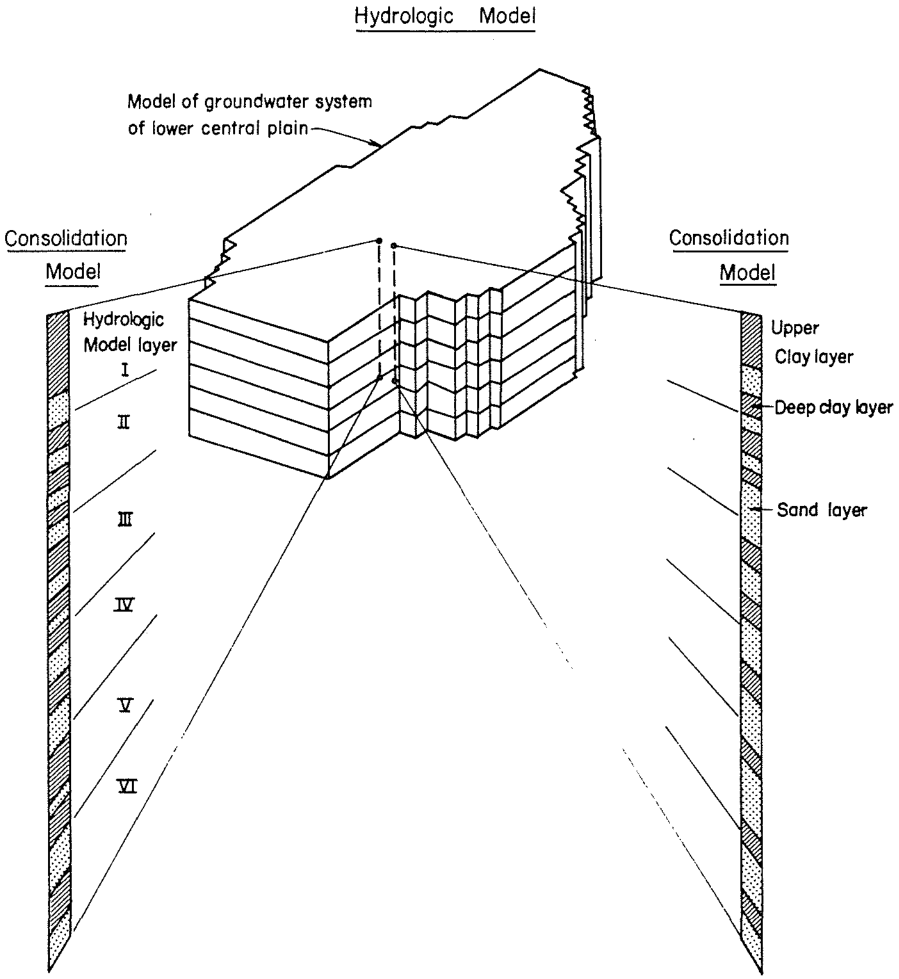


Fig. 6 Schematic Diagram of Coupled Model for Simulation of Land Subsidence Caused by Groundwater Withdrawal.

aquifers, and (ii) the water yielded from storage in the clay layers. The general expression for the term W_i can be given as:

$$W_i = \Sigma \frac{k}{\ell} \int_o^t M\ (t-\tau)\ \frac{\partial h'}{\partial \tau}\ d\tau \qquad ..... (2)$$

where k and ℓ are the permeability and thickness of the intervening aquitard respectively, M is a flow rate function involving a time rate factor c_v/H^2, h' is a function of piezometric levels in the given aquifer and two adjacent aquitards and t is the time since the start of simulation.

In a simplified case where the effect of time-delayed yield of the aquitard is neglected, the term W_i is given by,

$$W_i = \frac{k_{i-1}}{\ell_{i-1}} (h_i - h_{i-1}) + \frac{k_i}{\ell_i} (h_i - h_{i+1}) \qquad \ldots\ldots (3)$$

(b) Consolidation model - The Consolidation Model was developed to find (i) the detailed piezometric head distribution throughout the clay layer, and (ii) the magnitude of the compression of each slice, Δz, of the clay layer (due to the drop in piezometric head) and to sum these to represent the layer compression.

The governing equation for the piezometric head in a clay layer which takes account of the inhomogeneity of the clay, is:

$$\frac{\partial}{\partial z} k_z \frac{\partial h_c}{\partial z} = S_{sv} \frac{\partial h_c}{\partial t} \qquad \ldots\ldots (4)$$

which is equivalent to,

$$\frac{\partial}{\partial z} c_v \frac{\partial h_c}{\partial z} = \frac{\partial h_c}{\partial t} \qquad \ldots\ldots (5)$$

The piezometric heads in the aquifers derived from the Hydrologic Model are used as the boundary piezometric heads for the Consolidation Model for an individual clay layer at any location. The compression of each slice, Δz, caused by the piezometric head drop, $-\Delta h_c$, can be found from the e-log $\bar{p}$ relationship as follows:

$$\Delta\varepsilon = \frac{C_c}{1+e_o} \log \{1 + \frac{\Delta\bar{\sigma}}{\bar{\sigma}o}\} \qquad \ldots\ldots (6)$$

or alternatively:

$$\Delta\varepsilon = CR . \log \{1 - \frac{\Delta h_c \gamma_w}{\bar{\sigma}o}\} \qquad \ldots\ldots (7)$$

where $\Delta\bar{\sigma}$ is the effective stress increment which in subsidence study can be taken as: $- \Delta u = \Delta h_c \gamma w$; $\bar{\sigma}o$ is the effective overburden stress and $\Delta\varepsilon$ is the incremental strain for a soil slice of thickness Δz.

The calculation can be simplified by using the following relationship:

$$\Delta\varepsilon = -S_{sv} . \Delta h_c \qquad \ldots\ldots (8)$$

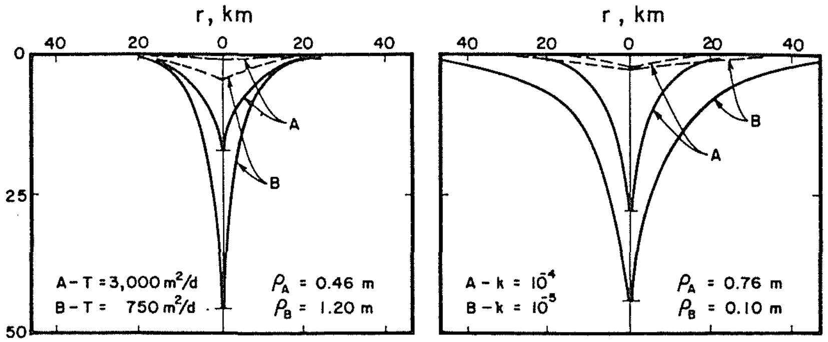
where S_{sv} (clay storage or clay compressibility) can be approximated without significant error as:

$$S_{sv} = 0.435 \frac{CR \gamma w}{\bar{\sigma}o} \qquad \ldots\ldots (9)$$

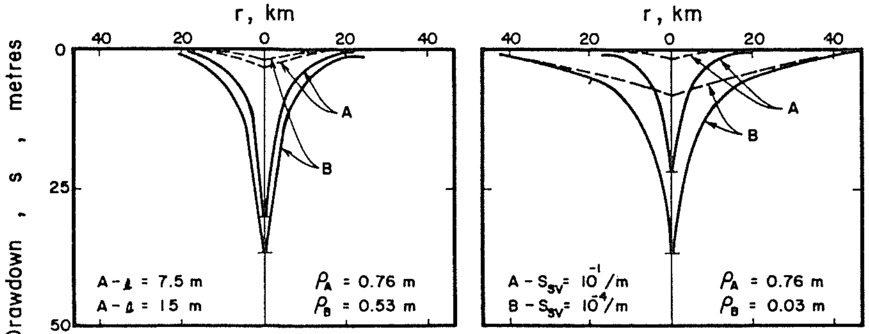
In the situation where piezometric head is rising instead of declining, the swelling (or recompression) parameters have to be used in place of C_c, CR and S_{sv}, due to the characteristic two moduli of compression of the soil. This is also the case where the effective stress at the moment is less than the maximum effective stress that the soil has experienced in the past. The change of parameter is also applied to c_v (k/S_{sv}) where the coefficient of consolidation should be changed to the coefficient of swelling, c_s.

The compression of sand layers occurs immediately when a pore pressure drop takes place due to groundwater pumping. The compression can be found directly from the pressure drop, the layer compressibility and the layer thickness. Summation of the compressions of all clay and sand layers through the depth constitutes surface subsidence at the location.

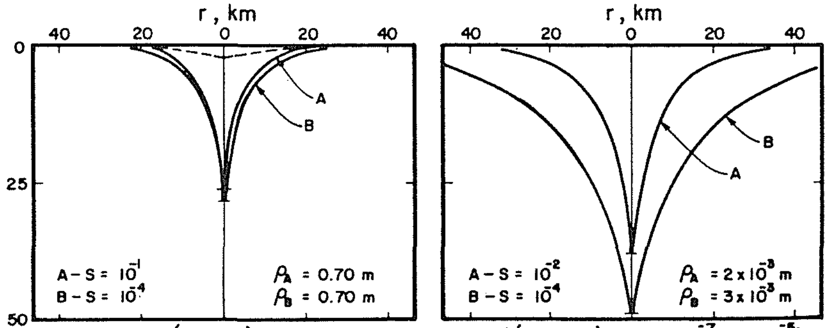
The parametric study was carried out by setting up a group of parameters to represent the 'normal' condition. Individual parameters were then varied



(a) Parameter T (b) Parameter k



(c) Parameter ℓ (d) Parameter S_{SV}



(i) 'Normal' case (ii) 'Isolated' case ($k = 10^{-7}$ $S_{SV} = 10^{-5}$)

(e) Parameter S

All for Well I in aquifer 2 after pumping period = 5 years

——— Drawdown in pumped aquifer

– – – – Drawdown in unpumped adjacent aquifers

ρ = Clay layer compression

Fig. 7 Effect of Various Parameters on Regional Drawdown Distribution (Normal Case at 5 years) to observe the changes in the solutions for the drawdowns in the aquifers and the magnitudes of clay layer compressions as compared to the solutions for the 'normal' condition. The parameters that were adopted for the 'normal' condition (these are approximately the parameters for the Lower Central Plain of Thailand) were:

i) transmissibility of the aquifers, $T = 1,500\ m^2/day$,
ii) storage coefficient of the aquifers, $S = 10^{-4}$,
iii) thickness of the clay layers, $\ell = 15$ m,
iv) permeability of the clay layers, $k = 10^{-4}$ m/day,and
v) compressibility of the clay layers, $S_{SV} = 10^{-2}$ /m.

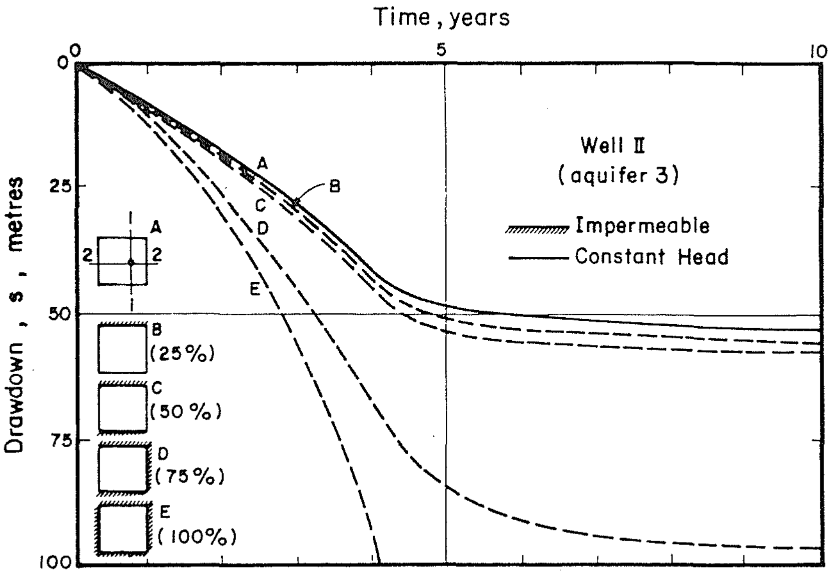
To assess the simulation in a rigorous way, full expression in integral terms was used for the term W_i in this parametric study. The technique in solving the resulting integrodifferential equation may be found from PREMCHITT (1981).

A summary of the effects of variation of the various model parameters in this assessment is presented in Fig. 7. The results are given for a time of 5 years after pumping started. The variation of the parameters affects both the magnitude and the pattern of piezometrical level drawdown over the domain. The compression of the clay layer adjacent to the concerned aquifer is also given in the figure. In the study of the parameter S, the variation within three orders of magnitudes (10^{-1} to 10^{-4}) did not produce significant changes in the solutions. However, when each aquifer is isolated and the yields from adjacent clay layers are reduced to insifnificant values ($k = 10^{-7}$ m/day, $S_{SV} = 10^{-5}$/m), considerable changes in the drawdown solution were observed (Fig. 7 (e)).

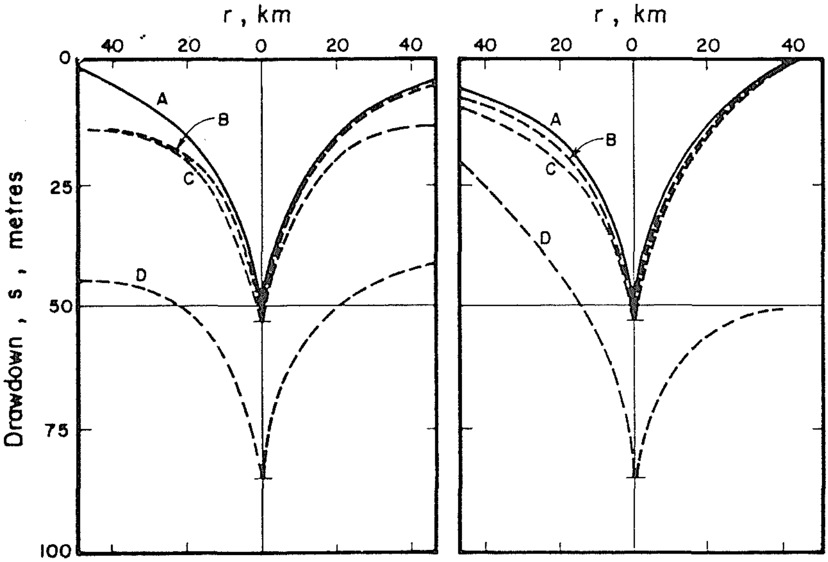
Influences of boundary conditions on the model responses were also assessed in this parametric study. Figure no. 8 shows the influence of an impermeable boundary on the maximum drawdown at Well II and on the regional drawdown distribution for the 'isolated' condition ($k = 10^{-7}$m/day, $S_{SV} = 10^{-5}$/m). The square domain was examined for the five conditions in which 100, 75, 50, 25 & 0% respectively of the periphery was in impermeable boundary, the remainder of the periphery was assigned to be a constant head boundary. It can be seen that the drawdowns are much greater for the cases where 100% and 75% of the boundary are impermeable than for the other three situations, in which the drawdowns are equal.

The influence of an impermeable boundary for the 'normal' condition is shown in Fig. 9 for the two extreme cases of an all-round impermeable boundary and an all-round constant head boundary. Surprisingly, there is almost no difference at all in the solutions obtained from the two schemes. This can be explained by the fact that the pumped groundwater comes mainly from direct seepage from the ground surface through the layers, together with yield from clay storage, rather than from recharge at the periphery for this 'normal' condition: it should be mentioned that the piezometric head in the upper clay layer was kept constant, which implies a continuous recharge from the surface. For the 'normal' case, the values of the parameter k (associated with the amount of vertical seepage through the clay layer) and the parameter S_{SV} (associated with the yield from clay storage) are a thousand-fold higher than for the 'isolated' case.

Of the parameters necessary for the mathematical simulation of natural subsurface strata for and subsidence predictions, some are commonly not accurately determined, some are determined by conventional tests which might not be representative of actual field behaviour and some either cannot be or have not been determined. It is important, therefore, to assess the relative significance of the various parameters involved in the model simulation. The parameters which have greatest influence should be carefully selected



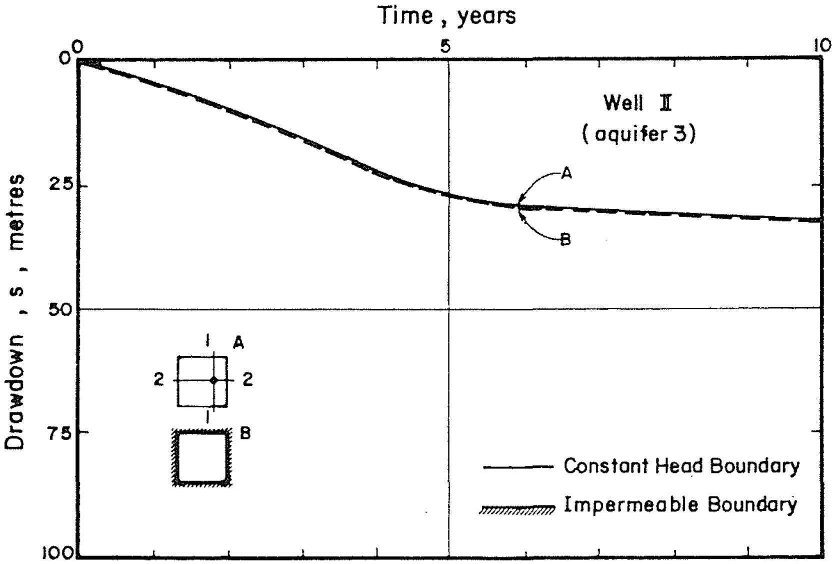
(a) Drawdown – Time Relationships



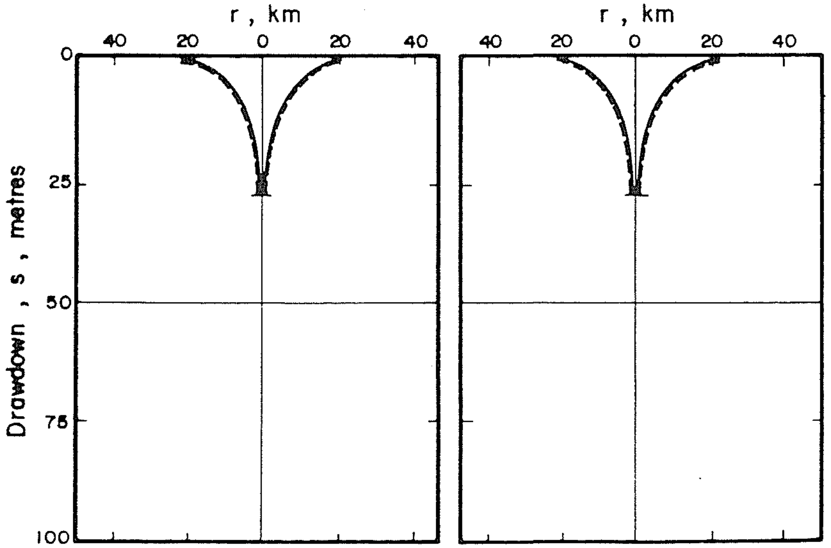
(b) Cones of Depression for Section 1-1 (5 years)

(c) Cones of Depression for Section 2-2 (5 years)

Fig. 8 Effect of Impermeable Boundary for 'Isolated' Case ($K = 10^{-7}$ m/d, $S_{sv} = 10^{-5}$/m)



(a) Drawdown – Time Relationships



(b) Cones of Depression for Section 1 – 1 (5 years)

(c) Cones of Depression for Section 2 – 2 (5 years)

Fig. 9 Effect of Impermeable Boundary for 'Normal' Case ($k = 10^{-4}$ m/d, $S_{SV} = 10^{-2}$/m) and require more attention than the other ones. The significance of each parameter in the Hydrologic Model can be observed from the changes in the solutions for drawdowns obtained from the model when each parameter is doubled or halved from the value under normal conditions. The relative significance of the parameters can be described on this basis as:

First order parameter - T (40%)
Second order parameter - k (15%), ℓ (12%)
Third order parameter - S_{sv} (4%), S (very small)

The value in the parenthesis is the change in the drawdown derived from the model when the 'normal' value of the parameter is doubled or halved.

Remedial Measures - Artificial Recharge

In order to increase the amount of water in the aquifers and to maintain the present piezometric levels, clean water must be recharged to the acquifers. If the amount recharged is greater than the amount withdrawn, the subsidence will stop and original piezometric levels eventually recovered.

Groundwater recharge by injection wells is relatively common and the technical problems relating to its application in Bangkok can be briefly outlined. Construction techniques are similar to those of pumping wells. To utilize the advantage of the method, the recharge well should penetrate to the heavy drawdown depth zone, i.e. from 100-200 m. It is expected that the recharge capacity of the well would be much smaller than the capacity of an equivalent pumping well (SUTER & HARMESON, 1960; BLAIR, 1970). Apart from pretreatment and mechanical cleaning to prevent clogging, the method of alternate pumping and recharging with a longer recharge period can also reduce the problems of clogging.

Another attractive recharge technique for the Bangkok area is the recharge through the base of some structures. Since these structures have to penetrate through the 15-25 m thick layer of soft and stiff Bangkok clay, the design and construction techniques in the Bangkok area, unlike the recharge well, have to be closely studied. The unsupported open-excavation is not feasible in the Bangkok area. The excavation with support structure to resist lateral earth pressure is possible but the shape and size of such a structure have to be selected in such a way that the amount of support structure is minimized and the construction technique is simple. The cylindrical pit seems to be the optimum choice, since there would be only the tangential compressive force on the members of the structure. Apart from this consideration, the site for the construction of the recharge structure should be selected in such a way that the recharge will be most effective in raising the ground-water level and that surface water is available in sufficient quantity. At the same time the amount of excavation should also be minimized by selection of an area where the top clay layer is thin.

A preliminary study on the feasibility of groundwater recharge in the Bangkok area has been carried out. Regarding the recharge pit, many sites in the area seem to suit the requirements. Three of these sites are selected for study. With the low groundwater level in the top aquifer prevailing in the area, the hydraulic head difference for the gravity flow of water from the pit is in the order of 15 to 20 m. For a pit diameter of about 10 m and the given head difference, theoretical estimation gives a flow rate through the recharge pit in the order of 5,000 to 20,000 m^3/day. Confirmation of this theoretical estimation can be made by an appropriate pilot test scheme. The pit of smaller size will require less excavation effort but the recharge rate will be reduced accordingly.

The construction of a cylindrical pit of 10 to 20 m in diameter which penetrates to 25 m depth is not a simple task, but it was found to be feasi-

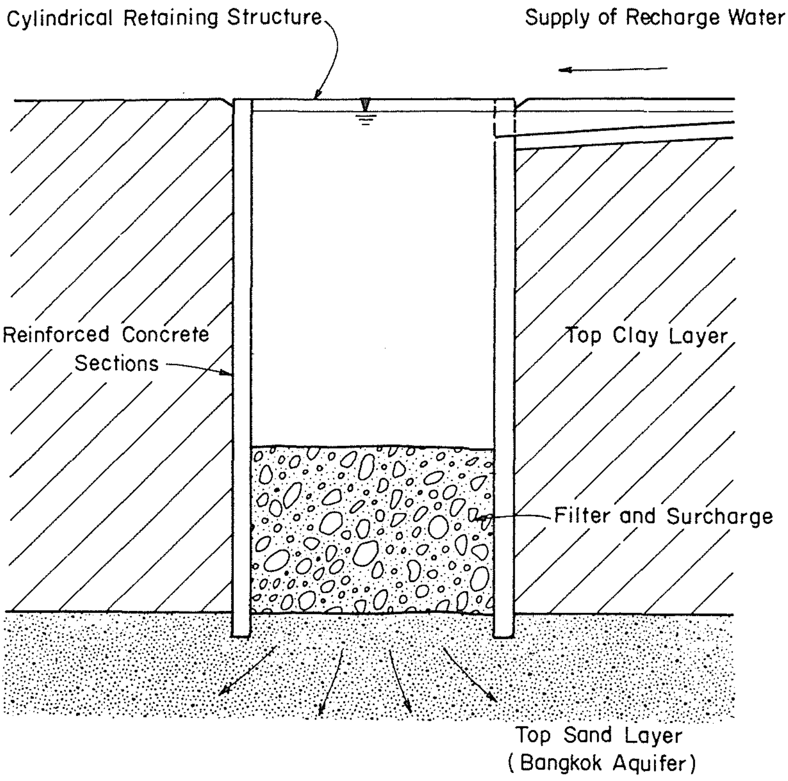


Fig. 10 Schematic Diagram of Recharge Pit

ble. Past experiences showed that construction of such a pit in the Bangkok area with the presently available techniques would take only 10 to 20 days. A schematic diagram of the proposed cylindrical pit is given in Fig. 10.

Quality of the recharge water is also an important factor in the recharge scheme. Recharge water of high turbidity may eventually clog the recharge structure and the aquifer. This will result in a significant reduction of the recharge rate. Some types of filter are necessary to reduce the amount of suspended solid particles and turbidity of recharge water before it will enter the aquifer. In the case of a cylindrical recharge pit, the filter can be placed in the pit or in the adjacent area or both. Regular cleaning of the used filter is also necessary. Other aspects of water quality also need attention since poor quality recharge water may pollute the existing groundwater system. The contents of such constituents as, dissolved oxygen, ammonia, bacteria and certain minerals are indications of undersirable water.

The performance of recharge schemes can also be assessed theoretically by mathematical model simulation. A detailed analysis will give a comprehensive picture of various aspects of recharge schemes. Research and development of the model in this regard has been initiated.

The actual implementation of recharge schemes including recharge pit and injection well recharge needs further extensive study and analysis. The only way to fully assess the potential of recharge schemes is to conduct a pilot test in the Bangkok area. Sufficient instrumentation to monitor field performance is necessary and it has to be carried out over at least a complete one-year period in order to assess the long term performance and the effects of seasonal variation.

Recommendations

The fact that Bangkok is sinking at a rapid rate must be realized and remedial measures should be taken at once. No time is left for indecision when 10 cm of ground surface is continuously subsided each year. Within the next 3-5 years, several square kimometer areas in Bangkok will be below mean sea level. The problem is not only complex but it is big also causing millions of Baht of loss annually. Since the ground water pumping will continue to increase and the city will continue to grow, the land subsidence problem will continue to grow unless we find a solution. In the light of this, the following recommendations are made:

1. An agency should be assigned to solve the problem and it must be given full authority to execute the approved plan.

2. The plan for the control of the land subsidence of Bangkok, Nonthaburi, Pathum Thani and Samut Prakan must be set such that the subsidence in these areas will be arrested within 5 years.

3. Details of these measures should cover the following:

3.1 The subsiding areas must be demarcated into zones according to the rate of subsidence. The critical zone of highest subsidence will require immediate remedial action.

3.2 Pumping of groundwater must be controlled, at the same time clean water should be recharged to the aquifers to prevent further drop of groundwater level and eventually to raise the groundwater level up to the original level.

3.3 Full support should be given to the MWWA Bangkok Water Improvement Program such that the full implementation of the program is on schedule.

3.4 Raw surface water in the equivalent amount of the present groundwater pumped to replace or replenish the groundwater must be allocated.

3.5 In planning the land use, the subsiding areas with its restrictions and problems must be taken into consideration.

3.6 Fees should be levied to the usage of groundwater at a comparable rate to the public water supply rate.

3.7 Monitoring and evaluations of results of the remedial measures must be done periodically so that modification of the plan to meet the objectives can be carried out.

3.8 The people who live in the subsiding area must be made aware of the subsidence problem due to groundwater withdrawal and they should actively participate/cooperate in the development of the remedial plan and action.

4. Adequate funds should be allocated by the government to solve this problem. Eventually this money will be recompensed through the groundwater pumping fees.

Acknowledgements

The authors express their sincere thanks to Dr. Piyasena Wijemunige and Dr. Premchitt Jerasak for their significant contribution as doctoral students to overall research in Bangkok subsidence which has been carried on for the last ten years. Mrs. Vatinee Chern and Mrs. Uraivan Singchinsuk did an excellent job in preparing the manuscript of the paper and the authors express their thanks to them.

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GROUND FAILURE CAUSED BY GROUNDWATER WITHDRAWAL FROM UNCONSOLIDATED SEDIMENTS--UNITED STATES

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Abstract

Aseismic ground failure is associated with regional land subsidence caused by ground-water withdrawal in at least 14 areas in 6 States in the United States. Two types of ground failure--tensile failures (causing earth fissures) and shear failures (causing surface faults)--are recognized. Fissures forming straight to arcuate patterns are caused by stretching related to bending of strata above a localized differentially compacting zone. Fissures forming complex polygonal patterns probably are caused by tension induced by capillary stresses in the zone above a declining water table. Surface faults occur along preexisting faults, many of which behave as partial ground-water barriers. Man-induced differential water-level declines across the preexisting faults have been sufficient to account for the heights of the historical offsets by a differential compaction mechanism. In the area most affected by surface faulting, Houston-Galveston, Texas, significant differential water-level declines across faults have not been observed and the specific mechanism of faulting has not been demonstrated.

Introduction

Aseismic ground failure, or ground rupture, is areally associated with most of the subsidence caused by aquifer compaction induced by ground-water withdrawal in the United States. Two general types of ground failure--tensile failures and shear failures--are recognized. Earth fissures result from tensile failures; opposing sides of the fissure move perpendicular to the plane of failure. Surface faults result from shear failures; opposing sides of the fault move parallel to the plane of failure. The principal hazard of ground failure is to engineered structures because deformation is localized. Accordingly, the principal economic effect has been in urban areas. In addition, earth fissures commonly are enlarged by erosion into steep-walled gullies and thus are a hazard to people or livestock. Earth fissures also can be detrimental to canals, levees, and dams because void space caused by extension during fissure formation creates the potential for catastropic release of water when fissures intersect these structures.

This paper summarizes the characteristics and the areal occurrence in the United States of ground failure associated with land subsidence and reviews current knowledge of the mechanisms that cause ground failure. For a more detailed and comprehensive summary, the reader is referred to Holzer (1984).

Ground-Failure Areas

Ground failure is both areally and temporally associated with ground-water withdrawal from unconsolidated sediment in at least 14 areas in 6

FIG. 1 Areas of ground failure associated with subsidence caused by ground-water withdrawal in the western United States.

States in the United States, all in the western part (Fig. 1). These areas are geographically widespread, and both the density of failures and the types of failure vary greatly from area to area; therefore, the generalized map (Fig. 1) does not show the local effect. For example, only single isolated failures have been reported in the Antelope, Santa Clara, and Yucaipa Valleys, California, and in the Raft River valley, Idaho. By contrast, ground failure is widespread in both south-central and southeastern Arizona, the Houston-Galveston area, Texas, and Fremont Valley, California. In the two combined areas in southern Arizona, for example, the total number of failures, producing predominantly earth fissures, is in the hundreds; in the Houston-Galveston area more than 86 historically active surface faults that have an aggregate scarp length of 240 km have been documented. Of the three largest subsidence areas in the United States, the San Joaquin Valley, California, south-central Arizona, and Houston-Galveston, Texas, only the latter two areas have significant numbers of recognized ground failure. Only four ground failures, resulting in one surface fault and three earth fissures, have been reported in the 13,500 km^2 San Joaquin Valley subsidence area.

Earth Fissures

Earth fissures are the most spectacular type of surface deformation associated with ground-water withdrawal (Fig. 2) because of their length

FIG. 2 Earth fissure, approximately 1 m across, in south-central Arizona that has been enlarged by erosion.

and the enlargement of the original tension crack by erosion. The longest fissure zone of those studied in the United States is 3.5 km long (Holzer, 1980b), and lengths of hundreds of meters are typical. Fissures commonly are enlarged by erosion into gullies 1 to 2 m wide and 2 to 3 m deep. Fissures usually are first noticed after erosion along them has begun as a result of heavy rainstorms. Measurements of separations in caliche, trees, and engineered structures indicate that fissure separations do not exceed a few centimeters; the maximum separation reported is 6.4 cm (Holzer, 1977).

Calculations of crack volume, based on the size of gullies formed along fissures, on small fissure separation, and on the field observation that most of the eroded sediment washes downward and is deposited in the tension crack, suggest that tensile failure may extend to depths measured in dekameters (Holzer, 1977). The greatest measured depth is 25 m (Johnson, 1980).

Earth fissures form two general types of patterns: 1) straight to arcuate and 2) polygonal. The former patterns are predominant in the United States. Locally, straight to arcuate patterns may be complex (Fig. 3), but they can still be distinguished from those formed by networks of closed polygons. The polygonal patterns are similar in map view to those formed by desiccation cracks in fine-grained sediment. Diameters of polygons commonly range from 15 to more than 100 m (e.g., Holzer, 1980b).

On the basis of contouring of regional geodetic data, i.e., the leveling of bench marks that have a spacing greater than 1 km, earth

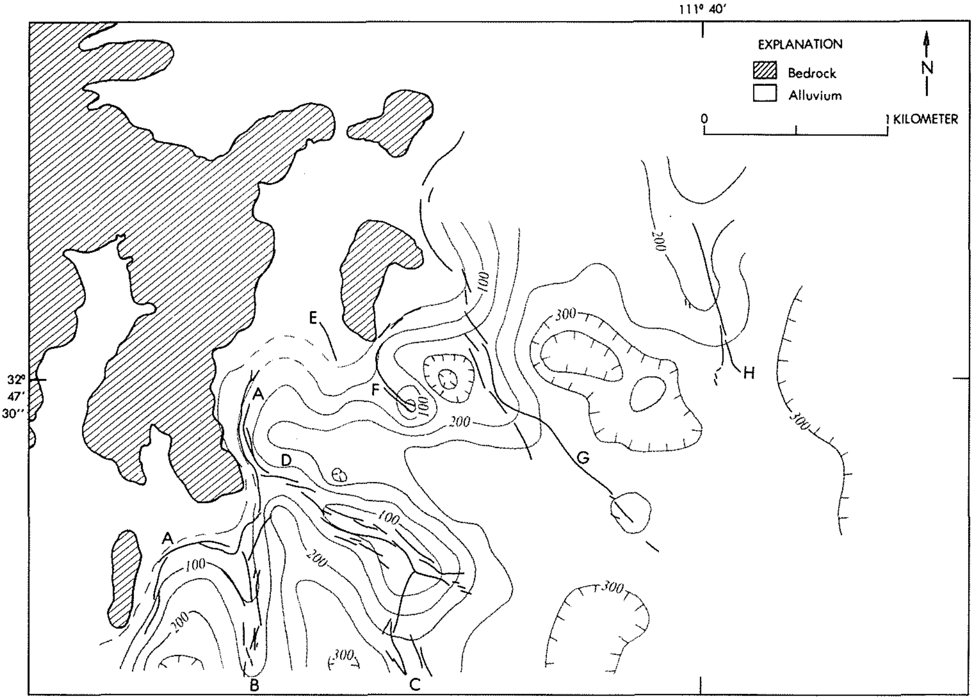

FIG. 3 Isopach map of unconsolidated alluvium beneath complex fissure area in south-central Arizona. Contour interval is 50 m. Hachures on contours point toward increasing sediment thickness. Fissures shown by dark heavy lines. From Jachens and Holzer (1982).

fissures occupy all positions and orientations within subsidence bowls. Analysis of surface deformation based on closely spaced bench marks near straight to arcuate earth fissures, however, indicates that these fissures are forming along zones of localized differential subsidence (Holzer and Pampeyan, 1981). In subsidence profiles oriented perpendicular to fissures, the fissures occur at the point of maximum convex-upward curvature (Fig. 4). Computations of horizontal strain from horizontal displacements of these closely spaced bench marks also indicate that horizontal tensile strains occur near the fissure and attain maximum tension at the point of the maximum convex-upward curvature (Holzer and Pampeyan, 1981; T.L. Holzer, unpublished data). Precise monitoring of the horizontal distance between bench marks spaced 20 to 30 m apart across fissures indicates that horizontal displacements continue as long as differential subsidence continues (T.L. Holzer, unpublished data). Jachens and Holzer (1982) described fissures in south-central Arizona that continued to open and accept sediment more than 25 years after their formation.

Although detailed studies of subsurface conditions beneath earth fissures have not been made in all the areas in which fissures have been reported, two types of subsurface conditions have been recognized in

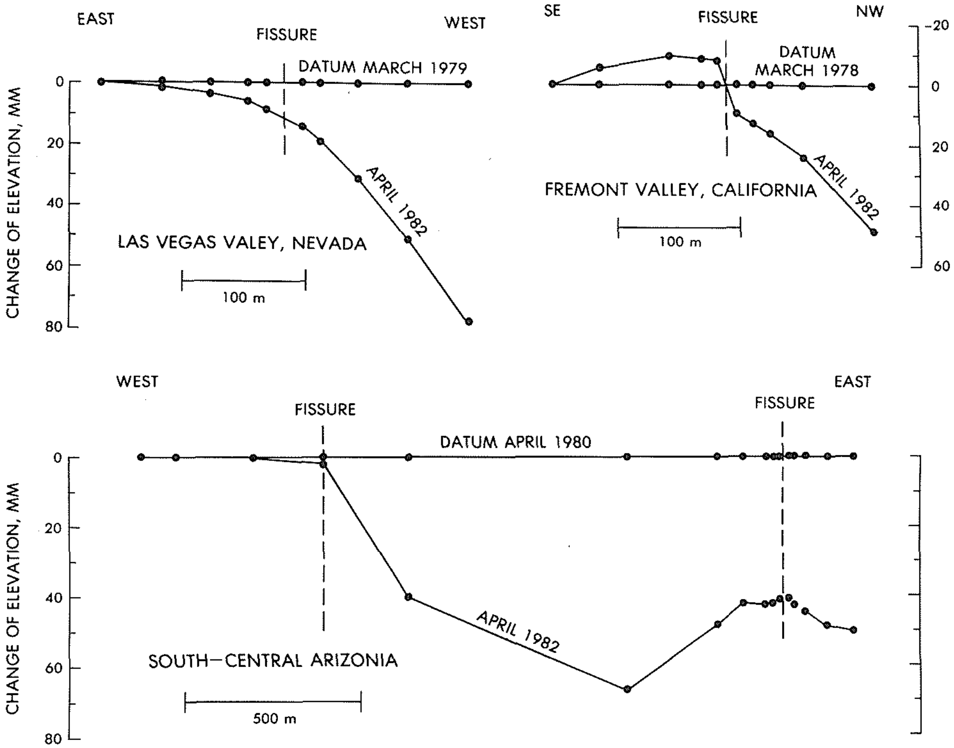

FIG. 4 Profiles of changes of elevations across earth fissures in Arizona, California, and Nevada. Decrease in elevation is positive.

areas studied. In south-central Arizona, most straight to arcuate fissures overlie zones of convex-upward curvature on the upper surface of the consolidated or crystalline bedrock that underlies the base of the unconsolidated aquifer system (Anderson, 1973; Jennings, 1977; Pankratz and others, 1978b; Jachens and Holzer, 1979, 1982). These zones range from ridges to "steps" in the bedrock surface. Figure 3 shows an example of bedrock control of a complex fissure system. In Las Vegas Valley, Nevada, and Fremont and San Jacinto Valleys, California, some fissures are coincident with preexisting faults. Subsurface characteristics of the fault zones, however, have not been investigated in detail. Gravity and magnetic surveys in Lucerne Valley, California, and southeastern Arizona, areas that have earth fissures in polygonal patterns, did not indicate special subsurface conditions beneath the fissures (R.C. Jachens, 1982, written communication). Fissures in both areas are underlain by fine-grained lacustrine and playa sediments.

Surface Faults

Scarps formed by faulting related to ground-water withdrawal generally resemble fault scarps of natural origin and can be confused with them. Faults suspected to be related to ground-water withdrawal commonly have

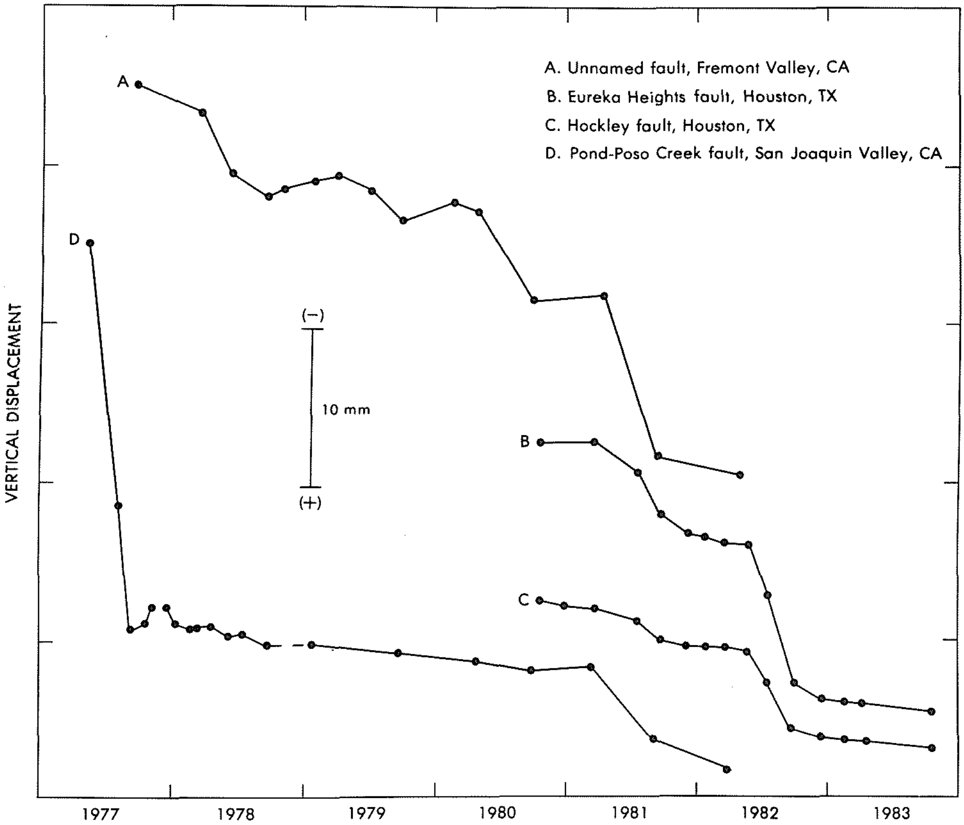

FIG. 5 Records of differential vertical displacements across surface faults in California and Texas. Variations in magnitude of seasonal displacement correlate with variations in magnitude of seasonal water-level fluctuation (not shown). For example, years with no or small displacement are years when seasonal water-level fluctuations were small. Increase in scarp height is positive.

scarps more than 1 km long and more than 0.2 m high. The longest scarp measured to date is 16.7 km long (Verbeek and others, 1979) and the highest scarp is 1 m (Reid, 1973). Both measurements were made in the Houston-Galveston area, Texas. Scarps range from discrete shear failures to narrow, visually detectable flexures, commonly along individual faults.

Surface faults generally grow in height by dip-slip creep along normal failure planes. Measured rates of vertical offset in the United States range from 4 to 60 mm year^{-1}. Neither abrupt movement nor seismicity has been reported in association with these faults. Monitored differential vertical displacements across these faults indicate that rates of offset vary over time. Although some short-term episodic movement has been reported (Reid, 1973), seasonal variations of offset that correlate both in magnitude and timing with seasonal fluctuations of water level are remarkably widespread (Reid, 1973; Holzer, 1978, 1980a; Holzer and others, 1983). Figure 5 shows examples of seasonal variation of fault offset. In addition, changes in long-term creep rates have been reported for a few faults (Van Siclen, 1967; Holzer and others, 1979, 1983). A striking example of these changes is in the Houston-Galveston area, Texas, where fault offset has ceased in an area where regional water-level declines have been reversed by reductions in ground-water withdrawal (Holzer and others, 1983).

Surface faults, like earth fissures, form at all positions and orientations within subsidence bowls (e.g., Holzer and others, 1983). Geodetic monitoring of closely spaced bench marks indicates that the land surface near the scarp may tilt above both the footwall and hanging wall blocks (Holzer and Thatcher, 1979; T.L. Holzer, unpublished data). Tilting is greatest near the scarp and has been observed to extend as far as 500 m from the scarp.

Surface faulting caused by ground-water withdrawal takes place along preexisting faults (Van Siclen, 1967; Holzer, 1978, 1980a; Elsbury and Van Siclen, 1983). Hydrogeologic studies of subsurface conditions beneath surface faults in Arizona (Holzer, 1978; Pankratz and others, 1978a) and California (Holzer, 1980a) have indicated that the preexisting faults are partial ground-water barriers across which water levels have declined differentially in conjunction with ground-water pumping. The water-level differences are in the same sense as the historical fault offset. In the Houston-Galveston area, Texas, evidence for differential water-level declines across faults is equivocal (see Kreitler (1977) and Gabrysch and Holzer (1978) for discussion).

Mechanisms

Subsurface conditions and surface deformation measured near straight to arcuate earth fissures, as well as theoretical considerations, indicate that these ground failures are caused by localized differential compaction. The fissures result from horizontal tensile strains produced by bending of the overburden. The strains attain maximum tension at the point of maximum convex-upward curvature in the subsidence profile (Lee and Shen, 1969). By modeling the bending process within a small area in south-central Arizona, Jachens and Holzer (1982) estimated that tensile strains at failure ranged approximately from 0.02% to 0.2%. These values agree with strain at failure inferred from average annual strain rates measured across earth fissures at other locations (Holzer and Pampeyan, 1981).

The complex polygonal network pattern of some fissures suggest that these fissures are caused by a horizontally isotropic tensile stress field. By analogy to desiccation cracks, the probable source of such tension is the large negative capillary stress in the dewatered zone above a declining water table. Such a mechanism was proposed by Neal and others (1968) to explain naturally occurring fissures that form giant polygons on playas.

Investigations of subsurface conditions and surface deformation near two faults in Arizona and California suggest that localized differential compaction can also cause modern surface faulting. Both the surface faults coincide with preexisting faults that are partial ground-water barriers. Man-induced water-level differences across the faults and inferred specific compaction of the sediment were sufficient to cause localized differential compaction across the preexisting faults equal to the observed scarp heights (Holzer, 1978, 1980a). Reid (1973) and

Kreitler (1977) have proposed that such a mechanism may also apply to surface faulting in the Houston-Galveston area, Texas, although the magnitude of water-level difference required to cause the offsets observed there does not appear to be compatible with available water-level data (Gabrysch and Holzer, 1978). This result does not preclude possible localized differential compaction. Many faults in the Gulf Coast commonly were active during deposition of the sediment that they offset because they affected sedimentation. Therefore, different thicknesses of compressible material may exist across these faults. If present, such localized lateral changes of thickness across preexisting faults might be sufficient to cause discrete differential compaction. In any case, the specific mechanism of faulting in the Houston-Galveston area has not been demonstrated despite strong circumstantial evidence linking historical faulting to water-level declines (Holzer and others, 1983).

Discussion

Ground failure takes place in most of the areas of land subsidence caused by ground-water withdrawal in the United States. Only a few subsidence areas do not have ground failure. The first failures in each area took place after subsidence began, and in those areas that now have large numbers of failures, the number gradually increased as subsidence continued. Thus, in a sense, ground failure may be considered as a secondary, although relatively common, condition caused by ground-water withdrawal from unconsolidated sediments.

Variations in the density of failures from area to area are conspicuous and are obviously determined by more than just areal differences of water-level decline and compressibility of sediments. For example, the areal extent and magnitude of subsidence in the San Joaquin Valley of California is the greatest of any area in the United States, but ground failure is rare. Part of the explanation for these density variations probably lies in differences in the subsurface conditions among the areas. Surface faults and straight to arcuate earth fissures are associated with preexisting faults and subsurface zones conducive to localized differential compaction, respectively. In areas like the San Joaquin Valley, these special subsurface conditions are rare (e.g., Miller and others, 1971).

Prediction of locations of potential ground failure on the basis of subsurface conditions appears to be feasible. However, it may not be economically feasible to acquire the detailed subsurface information for all applications. In some areas, inexpensive geophysical techniques may be satisfactory. For example, precise gravity surveys in south-central Arizona have been a practical means of delineating zones underlain by convex-upward curvature in the underlying bedrock surface (Jachens and Holzer, 1979, 1982). These surveys can delineate the potential fissure zones within a few dekameters. Jachens and Holzer (1982) also have shown that if sufficient data on tensile strength and the configuration and compressibility of subsurface materials are available, the finite-element method satisfactorily predicts the approximate magnitude of water-level decline at which fissuring will take place.

Finally, although no efforts to control ground failure have been attempted, investigations of relations between rate of fault movement and water-level change indicate control may be possible. In the Houston-Galveston area, Texas, and the San Joaquin Valley, California, fault movement stopped when water-level declines were reversed.

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Publication authorized by Director,
U.S. Geological Survey
March 14, 1984

EARTH-FISSURE MOVEMENTS IN SOUTH-CENTRAL ARIZONA, U.S.A.

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Abstract

Horizontal strain measurements of earth fissures have been made semi-monthly over a six-year period, at locations near the Sacaton Mountains, the Picacho Mountains, and in Avra Valley, Arizona, U.S.A. Permanent and temporary installations have been developed and perfected using dial gauges and transducers to measure horizontal earth and fissure movements. A glass tube is fixed on one side of a fissure and pushes against a dial gauge fixed to the other side. Installations are buried in shallow trenches in protective enclosures. During one month, discrete dial gauge measurements indicated a closing rate of 34 μm/day at a site in Avra Valley. During the same month, continuous displacement transducer measurements over a several-day period at the same site indicated a closing rate of 37 μm/day. These measurement techniques, which differed by two orders of magnitude in their precision, agreed within 90%. Among different fissure movements, the greatest total was 41.33 mm, the greatest single event was 31 mm, and, exclusive of that, the greatest net was 16.54 mm.

Preliminary analyses of data indicate certain trends. Fissures open and close repeatedly. Large earth-fissure movements have been smooth over long periods of time with occasional sudden jumps. With few exceptions, there appear to be no significant differences in the character of movements of old and new earth fissures. They tend to close after long, dry periods and to open after heavy rainfall. During a major event in a single locality, all fissures were affected -- some opened and some closed. Following a severe storm, one fissure abruptly opened 31 mm within a 14-hour period, while one nearby parallel fissure opened 1.6 mm, and another nearby parallel fissure closed 2.1 mm. A correlation appeared to exist among exceedingly high rates of ground-water extraction, rainfall, and earth-fissure opening when the pump rates exceeded a baseline of maintained high pump rates. An earth fissure closest to the area of the greatest amount of subsidence had the greatest total amount of movement. Infilling of one earth fissure with earth materials had no effect. On two separate occasions after infilling, the earth fissure reopened with time.

Introduction

Pumping of ground water has induced land subsidence and earth fissuring in at least seven ground-water basins in Arizona. Subsidence damaged wells and canals and forced an expensive rerouting of the main Central Arizona Project canal southeast of Phoenix. Fissuring has adversely affected farming, railroads, pipelines, canals, dams, highways, and urban development. Existing evidence suggests that many fissures extend through much of the unsaturated zone. Thus, a potential for direct contamination of aquifers exists. New fissures continue to appear as water levels continue to decline, and existing fissures continue to open and close. Damage from subsidence will continue to occur in basins subjected to prolonged ground-water overdraft. Hence, techniques which will aid in the prediction of the amount and location of subsidence and earth fissuring will be of practical importance.

The study described in this report has been primarily a field-oriented project intended to measure and analyze horizontal movements of earth fissures. Earth-fissure movement is defined as horizontal opening and closing of an earth fissure, not the widening or narrowing of an earth fissure due to erosion or slumping. This is a report on work in progress based on preliminary analysis of data. A large portion of the research was devoted to the development and perfection of permanent and temporary short-base extensometers which measure horizontal earth-fissure movement and earth strain. The installations were designed to be constructed simply and cheaply, to minimize thermal noise, to obtain small resolution, and to operate for many years. Discrete measurements of earth strain were made every two to four weeks over several years in order to determine the character of earth-fissure movement. The horizontal strain measurements of earth fissures were made at seven dial-gauge extensometer installations near the Picacho Mountains, the Sacaton Mountains, and in Avra Valley, Arizona (Fig. 1).

Field Area

The study area is within the Basin and Range Lowlands Physiographic Province of Arizona, a region characterized by large alluvial basins bounded by mountains of predominantly crystalline rock. The elevation of the basins' floors slopes gently to the northwest from approximately 550 to 420 m. The earth fissures studied are located on the alluvial plains between the central basin and the basin-bounding mountains.

The basins are underlain by thick, unconsolidated alluvium, anhydrite, and indurated conglomerate deposits, and are bounded by fault-uplifted mountains of plutonic, metamorphic, and volcanic rock. The sedimentary

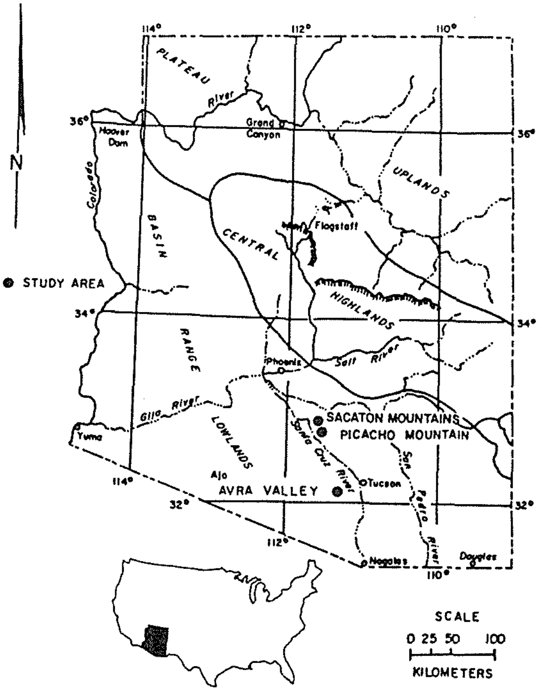

FIG. 1 Map of the study area.

sequence extends as deep as 3,000 m (Peirce, 1976), of which at least the upper 750 m is unconsolidated alluvium (Hardt and Cattany, 1965). Because there are nearby outcrops, most study sites are believed to overlie buried pediments, which is to say that the sites overlie or are mountainward of inferred basin-boundary faults. Depth to bedrock beneath the sites is on the order of 50 to 300 m. Geophysical investigations indicated that the local bedrock surface may be irregular (Pankratz, Ackermann, and Jachens, 1978; Jachens and Holzer, 1979). Tectonically, the region is relatively inactive.

The report by Hardt and Cattany (1965) provides a basis for interpretation of the Picacho basin ground-water system. Three hydrogeologic units were identified: lower and upper sand and gravel units separated by a unit of silt and clay. The three units extend to a depth of more than 750 m. The lower sand and gravel unit is a heterogeneous mixture of sand, gravel, and clay ranging up to 150 m in thickness, but is generally between 30 and 75 m thick. The middle silt and clay unit confines the lower unit ground water. In the absence of the silt and clay unit, the lower and upper units are indistinguishable and unconfined. Ranging in thickness from 0 to 600 m, the silt and clay unit is the least permeable and productive of the three units, but it does contain lenses of highly permeable, water-bearing material. The upper sand and gravel unit is similar to the lower unit lithologically and ranges in thickness from 15 to 185 m. This unconfined upper unit has the highest average permeability. However, the permeability varies both vertically and horizontally.

Natural ground-water movement in the Picacho basin is toward the northwest (Hardt and Cattany, 1965). Pumping from storage created local potentiometric surface depressions which now affect the direction of ground-water flow. Ground-water levels have declined as much as 100 m since 1923, nearly reaching the bottom of the upper sand and gravel unit. Production wells now reach to a depth of 760 m (Holzer, Davis, and Lofgren, 1979).

Subsidence in the Picacho basin has been attributed to hydrocompactión, increased effective stress due to loss of buoyancy, and basin readjustment (Christie, 1978). Subsidence prior to significant pumping and irrigation is most likely due to basin readjustment. Flood irrigation has caused hydrocompaction of unsaturated, near-surface alluvium which is rich in clay. This reasonably accounts for a portion of subsidence. Compaction of unconsolidated alluvium in response to the loss of buoyancy during decline of ground-water level due to ground-water extraction is believed to be the main cause of subsidence (Poland, 1967).

Earth Fissures

The earth fissures in south-central Arizona occur at the land surface as linear and curvilinear features which are generally segmented rather than continuous, in places forming an *en echelon* pattern. Generally hundreds of meters (but up to kilometers) in length, eroded earth fissures appear as open cracks in the ground surface and have dimensions of width and depth that range from centimeters to tens of meters. Initially only millimeters in width, they may be as much as 25 m deep (N. M. Johnson, written communication, 1979). The appearance of small depressions (termed "dimples") and hairline fissures on the ground surface prior to the opening of fissures and the observation of eroded portions of fissures beneath unruptured ground suggest that fissures may propagate upwards from depth. The diversion of surface-water flow and stream flow into fissures results in erosional enlargement. Laney, Raymond, and Winikka (1978) used the term "fissure gully" to describe eroded earth fissures. Because of the abrupt lowering of local base level produced by fissures, stream channels intercepted by earth fissures are subject to rapid gulleying.

Initially, fissures at the surface appear as hairline cracks with apertures of 1 mm or less. Following a heavy rain, small collapsed areas form in the soil along the former hairline crack. These small dimples, which may be from 4 to 60 cm in diameter, resemble animal burrows, but are aligned in many places for more than 100 m. Distances between dimples vary from meters to tens of meters. With time, some of these rows of dimples develop into well-formed fissures.

Once formed, a fissure is enlarged rapidly by erosion due to sediment-laden surface runoff and intercepted streams cascading down into the earth fissure. Erosional enlargement at the land surface produces a fissure gully which is commonly 1 to 5 m wide, 1 to 4 m deep, and hundreds of meters long. Occasional holes in the bottom of the fissure gully drain water and sediment down into the original deeper and narrower fissure.

The large amounts of water and sediment entering an earth fissure internally erode the fissure, eliminating areas along the fissure walls which had maintained contact. Tensile stresses are released and the consequent strain produces increased fissure opening at the surface. Presumably, this internal erosion of fissures and resulting opening explains why so many fissures are reported after periods of heavy rainfall.

Earth fissures in south-central Arizona associated with subsidence probably can be attributed to man-induced water-level decline. Major fissuring developed after ground-water levels had been lowered significantly. Indeed, the relation between ground-water pumping and subsidence in the Picacho basin has been clearly documented (Schumann and Poland, 1969).

The stress-producing mechanisms which develop during ground-water extraction are complex. Davis, Peterson, and Halderman (1969) were the first to show clear field evidence of short-term horizontal strain of the land surface caused by fluid movement in the saturated zone. Wolff (1970) suggested that fluid drag in the direction of water movement towards a well could produce horizontal strain. Lofgren (1971) proposed that lateral seepage forces could produce horizontal strain sufficient to form earth fissures. Bouwer (1977) proposed that the rotation of large, semi-rigid slabs of alluvium responding to subsidence could cause earth fissures. Shrinkage in the unsaturated zone may contribute to subsidence (T. N. Narasimhan, Lawrence Berkeley, oral communication, 1978). These and other mechanisms may act in concert with or without differential compaction to account for the large number of earth fissures in south-central Arizona.

Differential compaction of nonindurated sediments overlying irregular bedrock surfaces probably accounts for most of the major earth fissures seen at the surface. Jachens and Holzer (1982) obtained the best evidence for differential compaction from a detailed gravity survey. They demonstrated a high correlation between earth-fissure locations and local relief in the bedrock surface. Many fissures occur over local bedrock highs.

Instrumentation

This research resulted in the design and development of permanent and temporary, short-base extensometers to measure horizontal movements of earth fissures and earth strain with a resolution of ±0.5 μm. The design and installation procedures are detailed below.

Dial-gauge extensometers were installed in trenches dug across cracks (Fig. 2). A 0.5 m deep hole was augered in each end of the trench; a 1.5 m long, 19 mm diameter, mild steel rod was driven vertically into the bottom of each hole; and concrete was poured into the auger holes up to the bottom

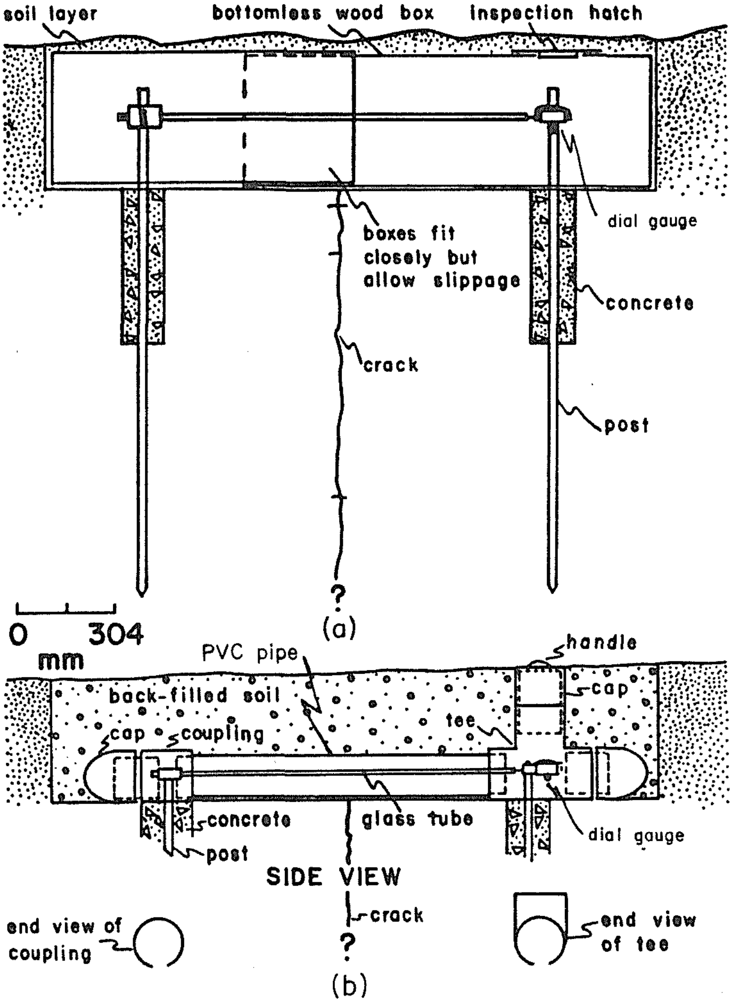

FIG. 2 Dial-gauge extensometer installation.

of the trench, leaving the rods projecting 50 to 150 mm into the trench. A 15 mm outside diameter, 1.0 to 1.3 m long, glass tube was epoxied into an aluminum holder attached to one rod by positioning the glass in the aluminum block, introducing 5 min. epoxy in the side access hole, and rotating the glass to draw the epoxy inside. Because of significant irregularities in the outside diameter of the glass, the hole for the glass was hand-reamed after drilling to assure a close fit without binding. After the epoxy solidified, the access hole and glass-to-metal contacts were covered with silicone rubber sealant for moisture protection. A dial gauge was attached to the other vertical rod so that the dial gauge's contact point pushed against a flat tip epoxied to the end of the glass tube. The dial gauges were Brown & Sharpe model 8231-941 or Starret model 25-881, with a range of 25 mm and a resolution of ±0.5 μm.

The dial gauge was backmounted or stem mounted, and the glass tube was initially positioned to allow for roughly 6 mm closing and 20 mm opening. The range of movement could be extended by changing dial gauge contact points. Earlier installations were enclosed by interleaving bottomless plywood boxes (Fig. 2a). Later installations developed by M. C. Carpenter were enclosed with 100 mm PVC (polyvinylchloride) pipe (Fig. 2b). The long section of pipe was put on before positioning and gluing the glass tube. The tee and coupling were slotted along the bottom so they could be slipped on around the posts from the ends. Silicone rubber was used to seal the pipe joints and the open slots around the posts. It was necessary to allow the rubber to cure completely before closing the installation because of corrosive gases produced in curing. A thermometer was left in each installation to measure temperature. Seven to ten readings were made by pulling back on the dial gauge rack and releasing it gently. The average reading was recorded. Usually, the range for all measurements was within the limit of resolution of the dial gauge. The dial gauge was enclosed in a plastic bag sealed with a rubber band around the stem. The dial gauge was removed from the fixture and the rack cleaned and oiled as needed.

Thermal noise was minimized by using materials with low coefficients of thermal expansion, controlling temperature fluctuations with insulation, exploiting the stability of soil temperature, and using temperature compensating design. Materials with low coefficients of thermal expansion such as fused quartz (5.8×10^{-7} $°C^{-1}$) were used as much as possible between centers of posts, and the use of high coefficient materials such as aluminum ($\approx 2.5 \times 10^{-5}$ $°C^{-1}$) was minimized. The method of epoxying minimized the amount of aluminum which could cause thermal effects while allowing the aluminum fixture to support the glass tube in spite of significant irregularities in the outside diameter of the glass. These irregularities caused problems with several other methods of clamping glass to metal. Tips of brass, aluminum, or copper could be used because of their short length. Burial of installations provided insulation and temperature stability.

A dial gauge installation was capable of measurements with a resolution of ±0.5 μm if the dial gauge was left in place. Establishing precise colinearity between the dial gauge rack and the glass tube improved resolution. Polishing tarnished dial gauge tips and brass or aluminum glass tube tips did not change readings, so corrosion was not considered important enough to change to stainless steel in any part of the installation.

It was possible to test the dead band of the measuring system by pushing or pulling with a shovel against a stake on the other side of the crack. After each stress, the dial gauge returned to its unstressed reading, indicating that the soil was elastic and the dead band was less than the resolution of the dial gauge. These tests were also performed with a small winch and cable system to test for binding in the PVC installations. These installations also appeared to give valid measurements for compressive and tensile strains with a resolution of $\pm 0.5 \times 10^{-6}$.

The installations were buried and camouflaged to eliminate destruction by people or animals, as well as to obtain favorable thermal environment. The plywood boxes were cheap and easy to install, but leaked, acted as solar stills, and were subject to termite damage. The PVC pipes were troublesome to install and expensive, but were sealable and were long-lasting. PVC pipe with a diameter of 100 mm provides barely enough room for adjusting the dial gauge or making readings.

A rough comparison with long-base extensometers lends validity to the horizontal strain measurements of the short-base, dial-gauge extensometers. Bench mark measurements at the Sacaton Mountain area agreed with dial gauge measurements (T. Holzer, U.S. Geological Survey, oral communication, 1978). This indicates that movement was horizontal and that rotational failure at the edge of the fissure was not a problem. Short-base extensometer measurements of fissured and nonfissured materials at Apache Junction, Arizona, showed the same trends and patterns (M. C. Carpenter, U.S. Geological Survey, written communication, 1980). This also minimizes the possibility of edge effects. Soil moisture is not thought to adversely influence the measurements. The general effect due to soil moisture is soil expansion which would simulate fissure closure. In general, fissures tend to open during wet periods (Fig. 3).

Conclusions

On the basis of a preliminary analysis of the data to date, the following observations and first inferences are presented. Please note that not all points have been discussed previously.

1. Permanent and temporary short-base extensometers have been developed to measure horizontal movements of earth fissures and earth strain with a resolution of ±0.5 μm.

2. Earth-fissure movement does not stop after the initial opening.

3. Measurements indicate that earth fissures in the study areas open and/or close over long periods of time (Fig. 3). Earth fissures tend to close after long dry periods. Earth fissures tend to open after heavy rainfalls.

4. A correlation exists among high pump rates, rainfall, and earth fissure opening (Fig. 4).

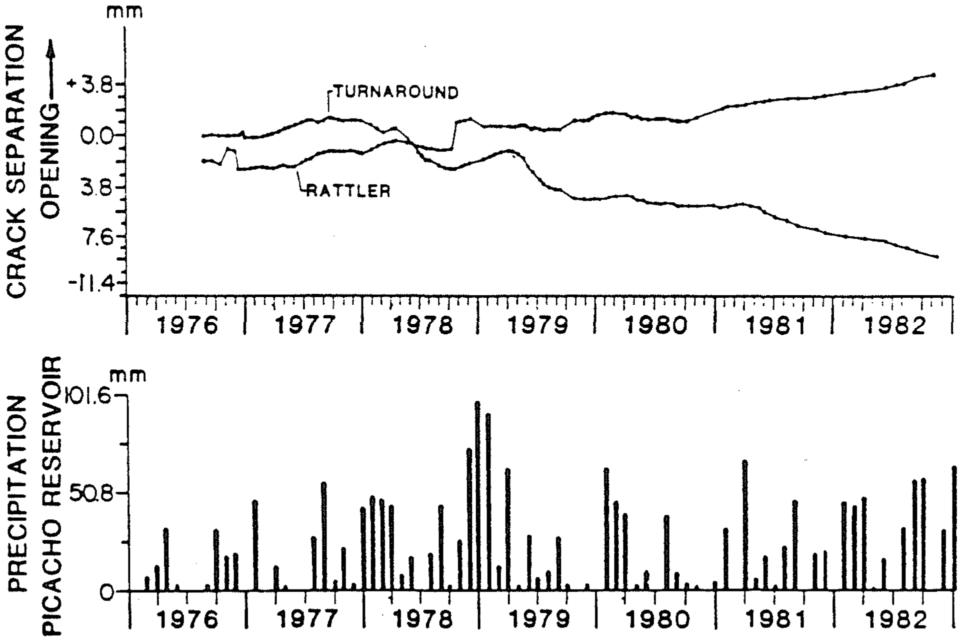

FIG. 3 Crack movement at Sacaton Mountains.

5. No difference in the character of movements of old and new earth fissures has been observed at this time (Figs. 3 and 5).

6. Data indicate that large fissure movements were smooth over long periods of time (Fig. 3).

7. The maximum rate of smooth earth fissure closing was 5.0 μm hr^{-1} over a 14-day period, and the maximum rate of smooth earth fissure opening was 4.1 μm hr^{-1} over a 17-day period.

8. Sudden movements occur occasionally. They usually follow heavy rainstorms (Fig. 4).

9. In Avra Valley, the dial gauge closing rate of 42 μm/day for the period January-March 1978 agrees very well with the transducer closing rate of 37 μm/day for 4-5 January 1978. Dial gauge measurements from 29 December 1977 to 28 January 1978 produced a closing rate of 34 μm/day. This represents an agreement of 88% and 92%, respectively, using two types of measurements which are overlapping and differ by two orders of magnitude in their precision.

10. On the evening of 23 July 1976, a rainfall of 31 mm caused an opening of approximately 25 mm of one earth fissure, while one nearby parallel earth fissure opened 1.6 mm, and another nearby parallel earth fissure closed 2.1 mm. This suggests that the desert alluvium has greater structural integrity than previously suspected. Perhaps rigid blocks of alluvium may exist between earth fissures (Fig. 5).

11. Artificial filling of earth fissures with earth materials has no lasting effect upon them. They simply reopen with time.

12. An earth fissure closest to the area of the greatest amount of total subsidence had the greatest total amount of movement.

Proposed Future Research

A major result of the research thus far has been the development of field installations to quantitatively measure horizontal movements of earth fissures and earth strain accurately and precisely. Future research must expand the present data base with the inclusion of ancillary measurements and experiments.

Future research should specifically include the following: a) discrete and continuous measurements of horizontal earth fissure movements and earth strain must be continued in present study areas and expanded to areas of different hydrogeologic characteristics; b) detailed water-level and land-surface subsidence records must be made; c) rainfall and soil moisture must be measured at selected sites; and d) more complex and detailed experiments must be devised and performed to test dial gauges used in the measurement of horizontal earth fissure movements.

Earth-fissure movements, water-level fluctuations, and subsidence will be analyzed and correlated using conventional and recently developed analytical procedures including spectral, harmonic, and time-series analyses. Empirical equations will be developed to express interrelations which may be discovered.

Acknowledgments

The work upon which this paper is based was supported in part by funds provided by the United States Department of the Interior, Office of Water Research and Technology, and the United States Geological Survey. I gratefully acknowledge the assistance of the City of Tucson and the University of Arizona Water Resources Research Center. I thank M. C. Carpenter for his help in the development of the short-base extensometers.

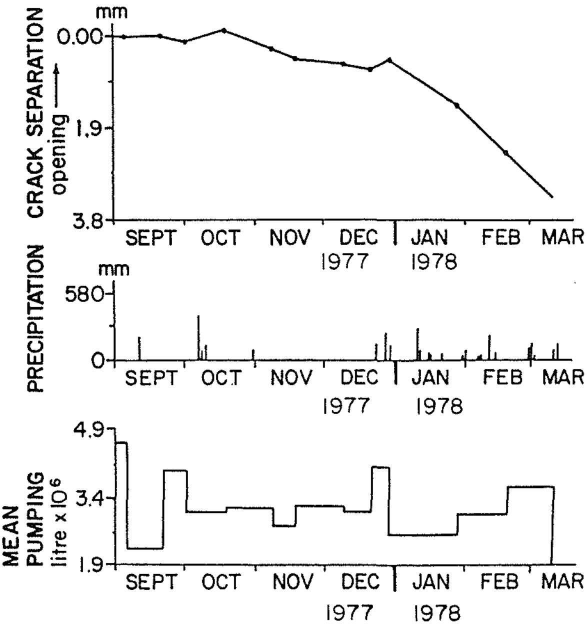

FIG. 4 Crack movement at Avra Valley well No. 7, September 1977-March 1978.

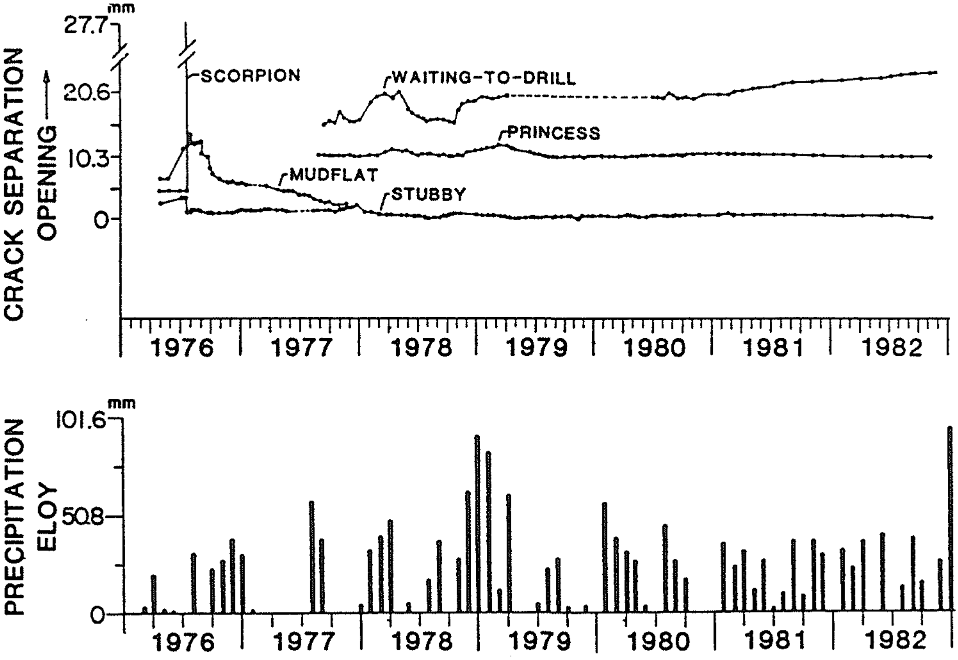

FIG. 5 Crack movement near Picacho Mountains.

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THE ROLE OF LAND SUBSIDENCE AND DAMAGE TO BUILDINGS IN THE SELECTION OF SUITABLE SITES FOR GROUNDWATER EXTRACTION

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Abstract

For groundwater management and planning purposes it is essential to gain a good insight into the consequences of groundwater extraction on the interests involved. The study as discussed in this paper deals with the hydrological effects of extraction, resulting in land subsidence and damage to buildings in the western part of the province of Utrecht. A great number of possible extraction sites in the area has been investigated and evaluated in order to select the most favourable sites.

Introduction

In The Netherlands, groundwater is a very important source for water supply. Because of favourable hydrological conditions, groundwater of a good quality is available in considerable quantities. Extraction of groundwater from an aquifer may cause a drop of piezometric head both in the pumped aquifer and also in the upper ones and will finally lower the groundwater table. In areas where the groundwater table is situated at shallow depth, these changes will have environmental impacts, in particular resulting in effects on the agriculture and the natural environment. Furthermore soil subsidence can be considered as an important limiting factor for groundwater exploitation, especially in areas where peat and clay occurs in the top layers, as is the case in the Dutch polder areas (Cramer, 1982).

During the period of 1974-1982, the Institute carried out an investigation in the western part of the Dutch province of Utrecht in co-operation with the public water supply company W.M.N. and the Water Board of the province of Utrecht. The main objective of the study consisted in the selection of potential sites for groundwater extraction from the deep aquifer (Hey, 1982). The interests to which attention has been given within the scope of the investigation are agriculture, in particular the crop yield of grass land, public water supply, natural environment and damage to buildings because of land subsidence. The application of techniques for predicting the effects of extraction on the interests involved and for comparing potential extraction sites leads to a number of optimal sites in the area in which there is the least impact of extraction on the interests involved.

Identification of the system

In order to study the consequences of groundwater extraction with regard to damage to buildings three aspects of the governing system will be distinguished, modelled and interrelated: the hydrological system, the geotechnical system and the building system.

The geohydrological situation demands a subdivision of the underground into two aquifers separated by a semi-pervious layer. A withdrawal of groundwater from the deep aquifer causes hydrological effects in the aquifers and in the semi-pervious layers. The drop of piezometric head in the shallow aquifer and the changes in the water table elevation are considered as the output of the hydrological system and are necessary as input to the geotechnical system.

Within the geotechnical system one can distinguish three processes in relation to land subsidence: oxidation, shrinkage and compaction. Firstly, oxidation as a result of dewatering can play a role in subsidence of soils with a high content of organic material, such as peat. However, experience has shown that the sensitivity to oxidation is reduced by the presence of a covering clay layer. In this case, where the cover has a thickness of about 0.50 m, oxidation of organic material will make a very small contribution to the land subsidence and will be neglected. The second phenomenon is a reversible process and a result of dehydration of the soil by evaporation. This process is not of any importance in this study. Thirdly, by lowering the piezometric head and the groundwater table, the pore pressure in the compressible holocene layers decreases and thus increases the effective stress in these layers. A result of this process is compaction and settlement of the soil. The Heidemij Consultancy Division which specializes in soil behavior, participated in the study as a contractor and calculated the geotechnical effects (Heidemij, 1981).

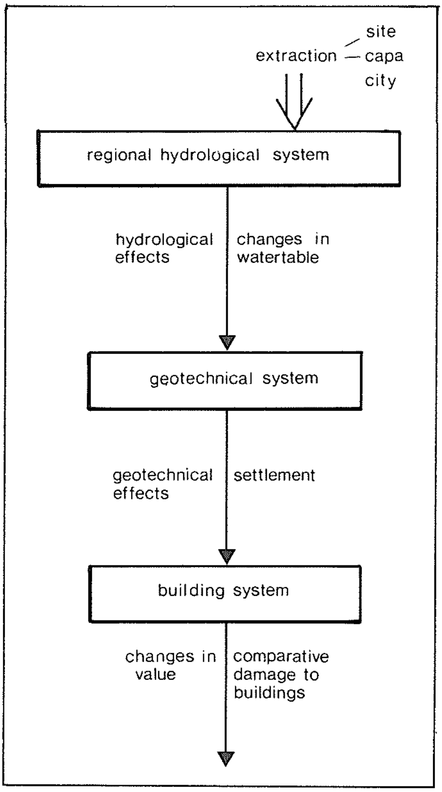


Fig. 1

The input to the building system consists of the settlement of the holocene peat and clay layers covering the investigated area. Finally the damage to buildings in the area is a result of the relation between the hydrological system in which the hydrological effects are calculated, the geotechnical system which determines the settlements and the system which determines the damage to buildings, as shown in figure 1.

The study of the hydrological system

The West-Utrecht area, about 400 sq km, is bounded in the south by the river Lek and in the east by two large canals. In the northern and western part, a brackish groundwater intrusion in the deep aquifer is considered as a boundary. The large artificially drained sub-sealevel polders have

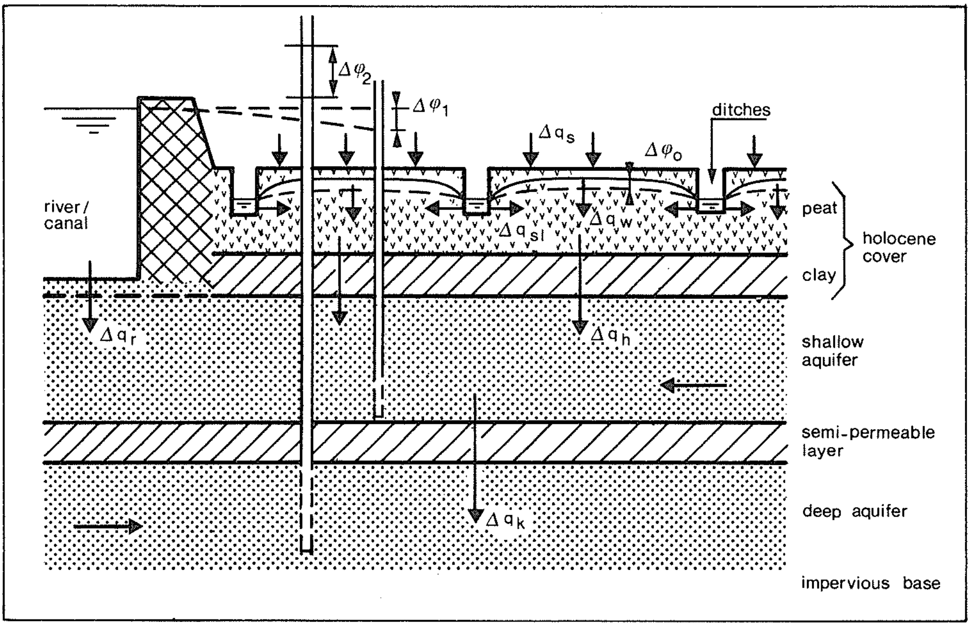


Fig. 2: Hydrological system and hydrological effects resulting from groundwater extraction from the deep aquifer.

mainly Holocene deposits (peat and clay) at ground level. The groundwater table lies about 0-2 m below surface. Differences in water level between the surface water system and the shallow aquifer create complicated flow patterns with alternating areas of upward seepage and infiltration. As a consequence, large volumes of groundwater are drained off by a multitude of ditches and discharged into main drainage canals.

As mentioned above, the hydrological system can be schematized as a two aquifer system, separated by semi-pervious layers. The hydrological base of the system, consisting of pliocene and miocene deposits which possess a high resistance against vertical groundwater flow, is found at about 150 m below ground level and is considered as impervious. The coarse grained sand layers of considerable thickness (about 60 m) forming the deep aquifer, are overlain by clayey sediments (thickness 10 to 40 m) which form the semi-permeable cover of the deep aquifer. The deep aquifer is known to possess a transmissivity of 2000 up to 4000 m^2/day. The hydraulic resistance (c) of the semi-permeable strata varies from 1500 up to 5000-7000 days. The shallow aquifer, with a thickness of about 30 m, has a transmissivity of 1500-2500 m^2/day and is overlain by about 20 m holocene sediments. Both aquifers can be considered as semi-confined. The selected hydrological system and the hydrological effects resulting from groundwater extraction from the deep aquifer are depicted in figure 2.

As a result of the extraction from the deep aquifer the piezometric head in this aquifer ($\Delta\phi 2$), the head in the shallow aquifer ($\Delta\phi 1$) and the water table elevation ($\Delta\phi 0$) will drop. Furthermore the flow through the semi permeable layers will change (Δqk and Δqh).

The replenishment of the shallow aquifer is taking place through changes in flow through the holocene cover (Δqh). Compensation of Δqh is occuring by:

1. A reduction or increase of horizontal flow of groundwater through the holocene top layer system. Because of the small transmissivity of the holocene layers, the changes in horizontal flow in these layers are negligible in comparison with 2. and 3.
2. A change in flow to or from the surface water system ($\Delta qsur = \Delta qdi$ for ditches + Δqr for rivers and canals).
3. A change in groundwater flow from the unsaturated to the saturated zone (Δqw), consisting of a decrease of evaporation (Δqs) and a change of storage in the unsaturated zone (ΔSu).

The process of compensation of Δqh can be expressed as:

$$\Delta qh = \Delta qsur + \Delta qw \tag{1}$$

where $\Delta qw = \Delta Su/\Delta t + \Delta qs$ (2)

and $\Delta qsur = \Delta qdi + \Delta qr$ (3)

When an extraction is begun, suppletion of groundwater will become available from changes in storage in the unsaturated zone (ΔSu). This quantity effects the shallow aquifer as a downward percolation in a short period of time (Δt). The decrease of water table elevation will continue untill there are steady state flow conditions and $\Delta qh = \Delta qsur + \Delta qs$. These flow conditions will be reached in a short period of time because of the possibility to keep the surface water system at a fixed level. Several studies learned that the decrease of evaporation (Δqs) is very small in comparison with $\Delta qsur$.
Equation (1) is reduced to: $\Delta qh \approx \Delta qsur$ (4)

The preceding discussion on the suppletion of extracted groundwater leads to the following assumptions:

- Compensation of extracted groundwater quantities from the deep aquifer is finally accomplished by means of an increase of seepage from the surface water system or a decrease of drainage;
- The hydrological effects can be computed if the relationship between the saturated groundwater system and the surface water system can be quantified;
- It is not necessary to link the unsaturated flow system (evaporation reduction) to the saturated system. Based on $\Delta\phi 0$, these calculations can be executed seperately;
- As for the fact that replenishment does not change with time, the hydrological effects can be calculated for a steady state situation.

Two dimensional, horizontal, steady state flow in an isotropic semi confined aquifer can be described with:

$$\frac{\delta}{\delta x}\left(kD_{(x,y)}\frac{\delta\phi}{\delta x}\right) + \frac{\delta}{\delta y}\left(kD_{(x,y)}\frac{\delta\phi}{\delta y}\right) = Q_{(x,y)} \tag{5}$$

where
$kD_{(x,y)}$ = the transmissivity of the aquifer in the point (x,y), in m^2/day.
ϕ = the piezometric head of the aquifer in m.
$Q(x,y)$ = the discharge or recharge of the aquifer in m^3 per day per m^2.

Equation (5) can be solved by using numerical techniques based on the finite element method. The hydrological effects caused by an extraction (see fig. 2) were calculated by means of the two layer groundwater flow model STATRECT (v.d. Akker, 1972), in which equation (5) is solved simultaneously for both the deep and the shallow aquifer. A grid of squares (1*1 sq km) with 448 nodal points was used.

The study in West-Utrecht covered 152 possible extraction sites and their influence areas (projects). On each site a withdrawal of groundwater from the deep aquifer of 6 million m^3 per year (190 l/sec) has been considered. The geometry, physical characteristics and the associated replenishment and extraction data of the investigated area were obtained from existing data or additional field measurements. The hydrological effects have been calculated successively in all projects concerned (concept of "walking pumping station"). The effects appeared to be restricted to an area of 7*7 sq km. This area has been called the standard influence area.

The geotechnical system

Due to lowering of the piezometric head in the shallow aquifer ($\Delta\phi 1$) and the consequent lowering of the water table ($\Delta\phi 0$), the pore pressure in the holocene layers will decrease. As a result, the effective stress in these layers increases, so that settlement of compressible clay and peat will occur.

The settlements have been calculated with the settlement theory of Terzaghi-Koppejan.

$$\Delta d = d\ (\frac{1}{c_p} + \frac{1}{c_s}\ \log t)\ \ln \frac{\bar{p} + \Delta p}{\bar{p}}$$

where:

Δd = the settlement
d = thickness of the layer considered
c_p = primary compression constant
c_s = secular compression constant
t = the time period after the piezometric head drop
$\bar{p}$ = the average effective stress in the layer
Δp = the increment of the effective stress in the layer

For the settlement calculations use was made of compression constants that were, unlike those in the usual theories, within certain limits dependent on the increment of the effective stress.

Settlement projections were made for undisturbed profiles to which no special loads are applied. The settlement of buildings on shallow foundations with a width of approx. 0.80 m, a bearing stress of approx. 5 kN/m^2 and a foundation depth of 0.80 m below ground surface, will be about 30% less than the settlement of the 'free' ground surface. This is caused by the relatively high effective stress under the foundation in the first metres of the profile.

The settlement projections covering the investigated area were based on existing information on profile structure and geotechnical properties of the various holocene layers.

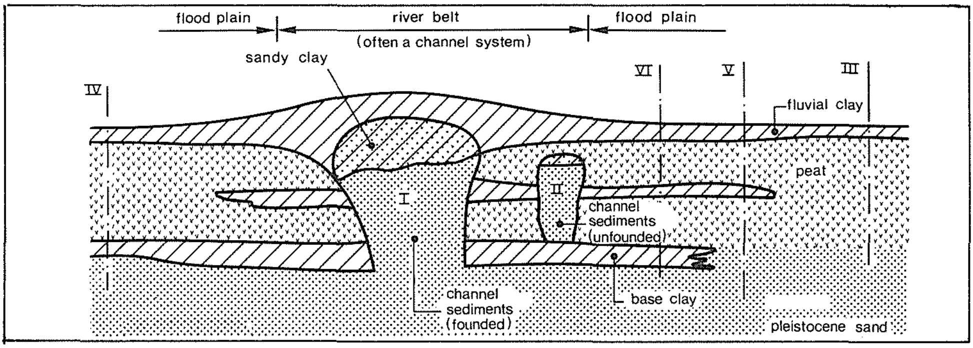


Fig. 3

In the area there are basically 6 profile types, represented schematically in figure 3.

On the basis of differences in thickness of the various layers and hydrologic differences, the profile types were detailed to 41 subtypes and

Table 1. Calculated final settlements (in mm) for the various subtypes of the profile types III and Vl at 10, 20, 30, 40 and 50 cm drop of piezometric head in the shallow aquifer (f. = favourable, av. = average, unf. = unfavourable)

Profile type and subtype	Calculated settlement in mm														
	Δϕl= 10 cm			Δϕl= 20 cm			Δϕl= 30 cm			Δϕl= 40 cm			Δϕl= 50 cm		
	f.	av.	unf.	f.	av.	unf.	f.	av.	unf.	f.	av.	unf.	f.	av.	unf.
III.Al	–	–	5	1	–	10	1	–	15	2	3	20	2	4	30
III.Bl	–	–	–	9	–	25	12	17	40	15	23	50	18	26	81
III.B2	–	–	–	9	–	25	12	17	40	15	23	50	18	26	81
III.Cl	4	9	11	8	18	21	11	28	31	14	48	55	16	56	67
III.C2	5	10	11	10	19	22	14	28	32	18	49	57	20	59	69
III.C3	4	8	10	7	15	18	10	22	27	13	40	48	15	54	59
III.C4	4	8	10	7	15	18	10	22	27	13	40	48	15	54	59
III.Dl	–	3	3	–	5	6	6	8	9	4	14	16	5	17	20
VI.Al	2	4	12	3	7	24	4	11	36	7	19	47	9	19	76
VI.A2	1	3	10	2	6	20	3	9	29	6	13	39	7	16	63
VI.A3	1	3	11	3	6	21	4	9	31	6	12	42	8	16	67
VI.A4	1	4	12	3	7	24	4	11	35	7	15	46	9	19	74
VI.A5	1	4	12	3	7	23	4	10	33	7	15	44	8	18	71
VI.A6	1	4	12	3	7	23	4	10	33	7	15	44	8	18	71
VI.A7	1	4	12	3	7	24	4	11	35	7	15	46	9	19	74
VI.Bl	1	2	6	2	4	13	3	6	20	5	8	33	5	10	41
VI.B2	2	3	7	3	6	16	5	8	25	7	12	39	9	15	50
VI.B3	1	2	6	2	4	13	3	5	19	5	8	31	6	10	38
VI.B4	1	–	6	2	–	12	3	9	18	5	14	30	7	17	37
VI.B5	1	–	6	2	–	12	3	9	18	5	14	30	7	17	37
VI.Cl	1	2	8	2	5	18	3	7	26	4	10	44	6	13	54
VI.C2	1	2	8	2	4	15	2	6	22	3	9	30	5	11	47
VI.C3	1	2	6	4	4	12	2	5	18	3	8	24	4	9	38
VI.C4	1	2	6	4	4	12	2	5	18	3	8	24	4	9	38

the area was divided into subareas that are more or less uniform as regards profile structure, thickness of the various layers and hydrologic conditions (map of soil subtypes).

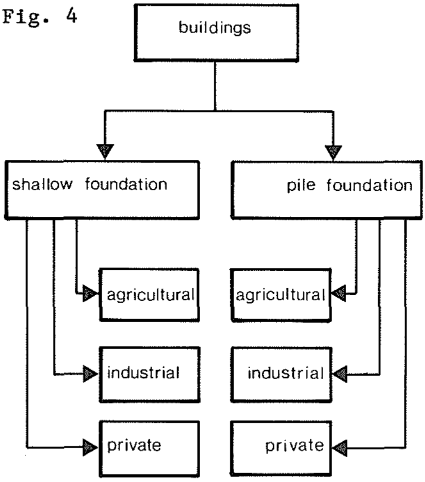
For each subtype, settlement calculations were made for the least settlement-prone profile (f.), the most settlement-prone profile (unf.) and the average profile (av.) within the subtype, taking into account a piezometric head drop in the shallow aquifer ($\Delta\phi 1$) of 0.10 m to 0.50 m. Table 1 illustrates the calculation results for the subtypes of the profile types III and VI. Profile type III is detailed to 8 subtypes and profile type VI to 16 subtypes.

The settlements that will occur in a standard influence area of a project can now be calculated using the map of soil subtypes, tabel 1 and the computed piezometric head drop ($\Delta\phi 1$) caused by a withdrawal of groundwater of 6 million m^3 per year in the project concerned.

The building system

The sensitivity of an area to damage to buildings was defined as the potential damage per unit of settlement. The potential damage because of settlement is dependent on the number of buildings in the area, the foundation type (shallow or deep), the use of the buildings and the amount of settlement. In order to compare 152 extraction sites and their influence areas it was necessary to make an inventory of the buildings present in the area. This was carried out according to the outline in figure 4. By consulting building experts it was learned that:

Fig. 4



1. On the average, the potential damage to private houses because of a given settlement is twice as great as the potential damage to buildings used for industrial or agricultural purposes;
2. On the average, the potential damage to buildings on shallow foundations is four times as great as the damage to buildings on deep foundations where the soil support is applied below the holocene layers (wooden or concrete pile foundations).

By combining the results 1 and 2 with the building inventory as shown in figure 4, the amount of various buildings in the area was transformed into standard building units with an equal sensitivity to damage.

The sensitivity is expressed in a weight-factor (vertical axis) which is calculated from:

$$wi = Ai / \sum_{i=1}^{152} Ai$$

where wi represents the weight-factor of project i and Ai the amount of standard building units in project i.

According to the hydrological effects, a standard influence area measures 49 sq km and consists of the extraction site in the centre (a model element of 1 sq km) surrounded by 48 squares of 1 sq km. By means of

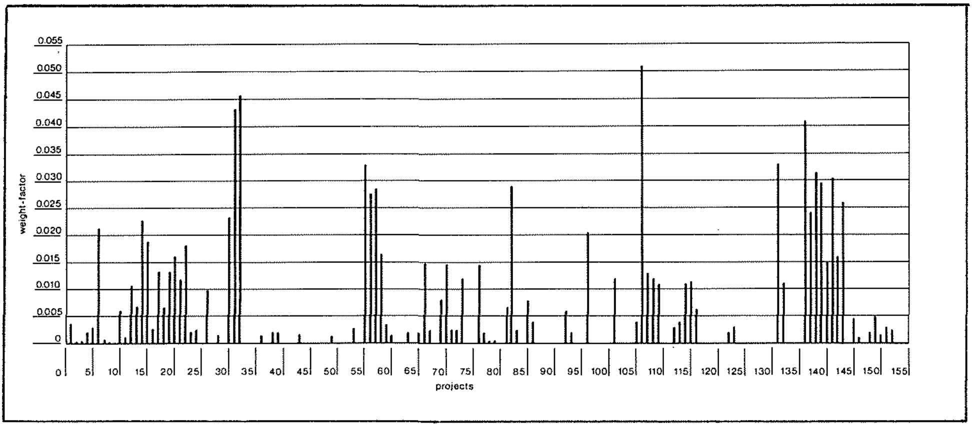
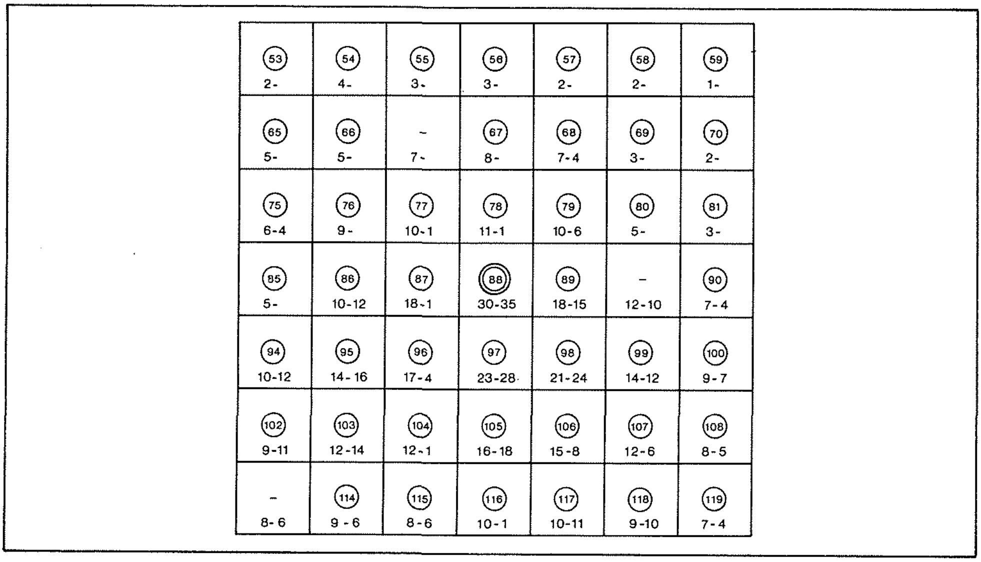


Fig. 5 shows the calculated comparative sensitivity to damage to buildings in those model elements in which 6 million m^3 per year can be extracted.

the hydrological flow model, the drop of head in the shallow aquifer ($\Delta\phi 1$) and the decrease of the water table elevation ($\Delta\phi 0$) were calculated successively for 152 extraction sites including their influence areas (projects). Taking into account the profile types and Table 1, giving the settlement per unit of $\Delta\phi 1$ per soil subtype, one can determine the settlement in the elements of the project concerned.

Figure 6 shows the results for project 88.



(53) 2-	(54) 4-	(55) 3-	(56) 3-	(57) 2-	(58) 2-	(59) 1-
- 5-	(66) 5-	- 7-	(67) 8-	(68) 7-4	(69) 3-	(70) 2-
(75) 6-4	(76) 9-	(77) 10-1	(78) 11-1	(79) 10-6	(80) 5-	(81) 3-
(85) 5-	(86) 10-12	(87) 18-1	(88) 30-35	(89) 18-15	- 12-10	(90) 7-4
(94) 10-12	(95) 14-16	(96) 17-4	(97) 23-28	(98) 21-24	(99) 14-12	(100) 9-7
(102) 9-11	(103) 12-14	(104) 12-1	(105) 16-18	(106) 15-8	(107) 12-6	(108) 8-5
- 8-6	(114) 9-6	(115) 8-6	(116) 10-1	(117) 10-11	(118) 9-10	(119) 7-4

Fig. 6. Standard influence area (project 88).

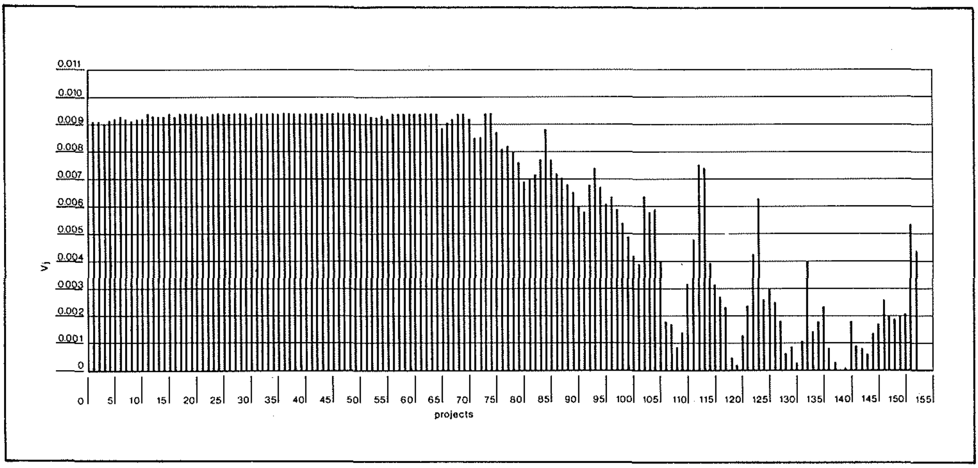


Fig. 7

(88) = number of project, defined as the number of the model element where a hypothetical withdrawal of 6 million m^3 per year is installed.

(97) = number of an element of project 88.

10-12 = drop of head in the shallow aquifer ($\Delta\phi 1$ = 10 cm) respectively the settlement (Δ d = 12 mm) of the holoce cover.

The potential damage to buildings of a project is proportional to:

$$D = \sum_{i=1}^{49} \Delta\, d_i * w_i$$ (= sum of products of settlement and weight factors in the influence area)

From the point of view of building conservation, a suitable choice of an extraction site can be made by comparing the value of D for the different projects (i.e D1, D2, ..., D152).

In figure 7 computed values of $v_i = 1/D_j$ (j=1,2,.....,152) are depicted after normalization. The best projects, where the damage to buildings is the least, have a high value of v_j.

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THE NATURAL SUBSIDENCE IN THE LAGOON OF VENICE, ITALY

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Abstract

Land subsidence of the Venetian territory is made up of two components: the first one due to natural causes and the other induced by man. The latter in particular was already studied in depth during the past years.

The natural process is due to the regional geo-tectonic conditions and to the progressive consolidation of fine-grained sediments. It should be stressed that natural subsidence has not been constant in time, because of the different depositional and erosional events.

On the basis of radiocarbon dating on organic remains, mainly peats and shells, it has been posible to estimate a 1.3 mm/y mean compaction (accumulation) rate during the Late Pleistocene (from 40,000 to 22,000 yrs b.p.) ; a first Holocene rate (7,000-6,000 yrs b.p.) of about 3 mm/y follows, and a noticeable decrease of velocity is then observed.

The recent quiescence of natural subsidence can be attributed to the reduced alluvial yield inside the Lagoon, after the diverting into the sea of the major tributaries. This hydraulic intervention was started by Venetians in the 14th century.

1. Introduction

In the area of the Po Valley, the phenomenon of natural subsidence assumes different values from the axial to the border zones. The lagoon area, situated on the northen side of the plain, presents values inferior by a factor of about 3 with respect to the axial zone (e.g. area of the Po River Delta). In evaluating natural subsidence it has not been possible to separate the component due to the lowering of subsoil, for structural-geological reasons, from that of consolidation of recent fine-grained deposits, due to geostatic load, because of the very minor incidence of this component. The evaluation of the rate of sedimentation (land subsidence) has been made by means of radiocarbon dating with C^{14} method, of both animal (shells) and vegetative (peat and wood) remains. Assuming that the sedimentation rate of these remains is the same as the deposits containg them, one can trace the average rate of sedimentation per unit of time, once the age and depth are known.

The field of investigation of this method, related to the detection range of C^{14}, is about 40,000 years, which places the investigation, from the chronological point of view, between the final phase of the last Würm glaciation and the present time.

The data used in the present work derive in part from published radiometric datings (Bortolami et al., 1977) from which only those referring to the lagoon area have been evaluated, and from a large number of new C^{14} analy-

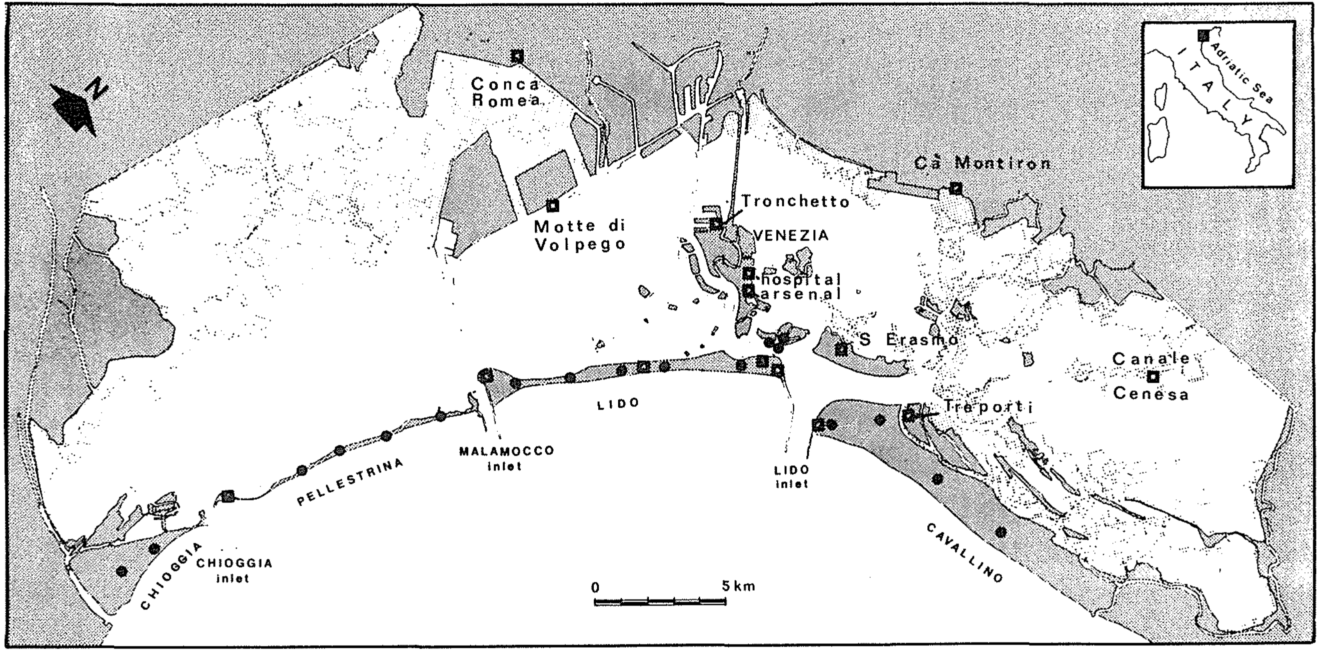

FIG. 1 - Location of boreholes from which samples for radiocarbon dating have been collected.

▣ : after Bortolami et al., 1977

● : after Gatto P., 1980

TABLE 1 - Description of samples from boreholes shown in figure 1.

No	Locality	Depth vs ground surface (m)	Depth vs at sea level (m)	Age (years B.P.)	Type
1	Sant'Erasmo	39.6	37.8	33.750 ± 1000	P
2		36.0	34.2	30.530 ± 800	P
3		19.0	17.2	19.250 ± 300	P
4	Treporti	41.3	40.4	35.650 ± 1200	P
5		34.6	33.7	29.320 ± 800	P
6		27.5	26.6	22.440 ± 500	P
7	Lido di San Nicolò	48.9	46.1	39.460 ± 1500	P
8		30.5	27.7	24.430 ± 400	P
9	Lido di Pellestrina	4.8	3.8	2.920 ± 100	OM
10	Ca' Montiron	12.3	10.4	22.090 ± 400	CM
11	Venice Tronchetto	45.7	43.7	36.450 ± 450	P
12	CNR 1 test hole	37.5	35.5	34.800 ± 500	P
13		24.3	22.3	21.750 ± 730	OM
14		6.6	4.6	4.350 ± 150	S
15		5.4	3.4	4.150 ± 70	S
16	Venice civil hospital	9.8	9.0	20.060 ± 300	OM
17	Venice civil arsenal	46.9	45.9	38.220 ± 1200	P
18	Alberoni	33.5	31.0	28.360 ± 800	P
19	Punta Sabbioni	18.0	17.8	20.000 ± 400	P
20		12.0	13.0	5.900 ± 120	S
21	Conca Romea	13.2	11.8	20.950 ± 400	P
22		6.4	5.0	19.410 ± 800	P
23	Lido of Venice	12.9	12.4	5.900 ± 100	S
24		10.1	9.6	5.490 ± 100	S
25		7.5	7.0	4.910 ± 100	S
26		1.9	1.4	3.340 ± 80	S
27		0.7	0.2	2.340 ± 90	S
28	Lido of Venice (north)	22.4	20.4	4.930 ± 100	S
29		18.6	16.6	4.840 ± 100	S
30		5.4	3.4	3.870 ± 90	S
31		5.6	8.1	2.020 ± 100	S
32	Motte di Volpego	23.0	24.7	23.450 ± 500	P
33		10.6	12.3	19.620 ± 315	P
34		5.0	6.7	1.700 ± 132	S
35		4.4	6.1	1.708 ± 85	S
36	Pellestrina	7.05	4.25	1.290 ± 110	S
37		19.20	18.4	7.150 ± 200	S
38	Lido	6.90	4.3	4.300 ± 200	S
39		8.90	7.55	5.300 ± 200	S
40		11.60	10.30	5.350 ± 200	S
41		17.40	16.15	21.700 ± 500	P
42		28.10	26.9	30.000	P
43		14.85	13.2	16.400 ± 500	P

TABLE 1 : continued

No	Locality	Depth vs ground surface (m)	Depth vs at sea level (m)	Age (years B.P.)	Type
44		17.20	15.75	20.900 ± 750	P
45		25.60	24.1	22.500 ± 950	P
46		9.60	7.55	4.700 ± 150	S
47		11.38	9.45	5.750 ± 170	S
48		25.00	23.2	22.200 ± 900	P
49		29.60	27.85	27.900 ± 1700	P
50		10.30	6.3	4.820 ± 150	S
51		22.55	18.6	21.000 ± 300	P
52	Sant'Andrea	16.50	14.95	24.000	P
53		17.90	16.35	22.000 ± 1000	P
54		8.40	6.9	5.970 ± 100	S
55		15.03	13.6	19.500 ± 200	P
56		16.92	15.45	20.000 ± 250	P
57		18.00	16.5	22.200 ± 1000	P
58	Cavallino	15.32	11.4	21.600 ± 650	P
59		28.52	24.65	24.500 ± 1000	PC
60		7.78	6.65	4.700 ± 100	S
61		18.90	17.8	22.900 ± 800	PC
62		10.50	9.8	5.410 ± 70	S
63		13.05	12.4	5.320 ± 80	S
64		15.40	14.8	7.950 ± 50	S
65		21.15	20.5	23.000 ± 1000	P
66		12.75	12.05	4.760 ± 150	S
67		16.55	15.9	20.500 ± 800	PC

Radiocarbon dates: P = peat ; OM = organic material ;
CM = carbonate mud ; S = shell ;
PC = peat in clay

Data 1 through 35 are excerpt from Bortolami et al. (1977).

ses performed on material derived from 18 continuous core boreholes carried out along the Venetian littoral (Gatto, 1980) (Table 1 and Fig. 1).

2. Geological and Paleogeographical Setting

As previoulsy pointed out the lithostratigraphical sequence under examination comprises the terminal part of the Upper Pleistocene and the Holocene.

From the depositional point of view, the pleistocene sediments are of continental environment and are made up of sands, silts and clays with peaty intercalations, of fluvial, lacustrine and marsh origin.

From the climatic point of view, in this time the precipitations are stored in solid form and the continent is covered by a thick cap of glaciers; consequently the sea level is lower by some tens of meters with respect to the present level.

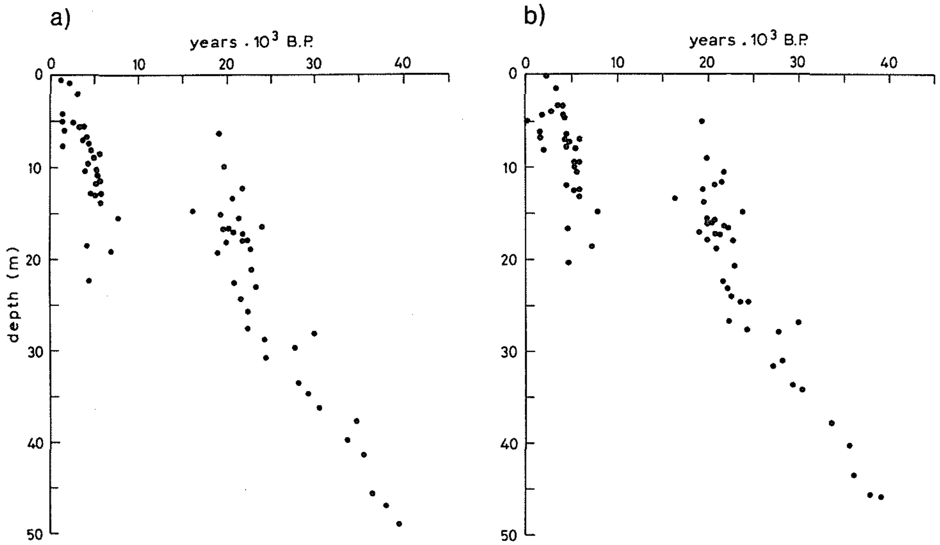

FIG. 2 - Radiocarbon ages against depth with respect to ground level (a), and mean sea level (b).

The Upper Adriatic, characterized by shallow sea beds, is transformed into a single, large paleoplain on which the continental waterways follow their courses in a sedimentary environment characterized by low energy.

In the cataglacial of this last Würmian phase, on the paleoplain a silty-clayed carbonatic cement paleosoil, locally called "caranto", was formed, and it is an overconsolidated layer with high geotechnical characteristics (Fontes and Bortolami, 1973; Gatto and Previatello, 1974).

With the warming of the climate, the continental glacial masses melt, the average sea level rises and invades the Adriatic paleoplain with, at first littoral, and lagoon deposits, until reaching more or less the present shape.

The caranto seals the Würmian paleoplain and therefore has a high stratigraphic significance, inasmuch as it represents the Pleistocene-Holocene limit (Gatto and Carbognin, 1981).

3. Data Analysis

The distribution of the radiocarbon datings relative to these continental littoral and lagoon deposits, show an excellent correspondence between age and depth, referred to both sea and ground levels (Fig. 2).

The following short comments refer to Fig. 3 where the used reference is the sea level.

- 40,000 ÷ 22,000 years b.p. There is a regular trend in the sedimentation rate controlled by subsidence of an average value of about 1.3 mm/y.
- 22,000 ÷ 18,000 b.p. There is an abrupt increase in the rate of sedimentation with a value of about 5 mm/y. This span of time corresponds to the maximum glacial crisis accompanied by higher load of glaciers on the continent to which corresponds the maximum effect of isostatic lowering.

- 18,000 ÷ 7,000 years b.p. There is a lack of sedimentation, or erosion, and the long exposure of the silty-clayed surface of the Adriatic paleoplain determines the formation of the caranto. In this period, with the bettering of climate and the melting of the continental ice, there is an increase in the hydrographic network with the starting up again of the erosive activity by the water-courses, which cut into the paleoplain noticeably. At the same time the level of the sea begins its progressive rising, giving origin to Flandrian transgression on the continent.
- 7,000 ÷ 5,000/4,000 years b.p. There is the depositing of littoral sands with abundant shell remains which fill the preceding erosion ditches and, in fact, the oldest Fladrian deposits are found, on the basis of available data, at 20 meters below the most recent Pleistocene ones. The first transgressive deposits date back about 7,000 years. In this phase, the rather high speed of sedimentations is about 3 mm/y and is controlled by the morphology of the sea bed.
- Starting from 4-5,000 years b.p., probably corresponding to the change in lagoon conditions similar to the present ones, the rate of sedimentation decreases preogressively to values around 1 mm/y, and less.

This overall picture is in agreement with that already outlined by Bortolami et al. (1977) even examining a wider area. Morover, for the most recent period the above described trend of land subsidence is confirmed by protohistoric and historic data. Beneath the Venetian area, in different sites along the Grand Canal, at Torcello island, etc., and at different depths varying between 6 and 4 meters below ground level, various remains referring to prehistoric eras from the Neo Eneolithic to the Second Iron Age have been found (Leonardi, 1960; Miozzi, 1969). The depths of the finds and the epoch which the remains belong, give a subsidence rate of about 1 mm/y. Further historic data referring to the Roman and paleo-Venetian ages (floors, bases of columns, etc.) confirm the above (cf. photos in Carbognin et al., 1984).

It must be pointed out that the most recent man-induced contribution has been considered in the quantative estimate of the natural subsidence process for the protohistoric and historic periods.

Concerning the present century, the rate of natural subsidence has been computed either on the basis of differences in elevation of several benchmarks for the period 1900-1930 (that is before groundwater withdrawals), or by a comparison of mean sea levels between Trieste, which is a "stable" coast and Venice: a rate equal to about 0.4 mm/y has resulted (Gatto and Carbognin, 1981).

Conclusions

Natural subsidence, assuming a linear trend, indicates an average of 1.2-1.3 mm/y for the whole interval of time considered. In spite of the legitimacy of the linear assumption, this is clearly interpolative of an undulatory trend related to periods of over sedimentation alternating with periods without deposition or erosional, as is particularly evident in the time interval 18,000-7,000 b.p. (see Fig. 3), which are placed in a wider already well-studied geoclimatic context.

It is possible that even the present slowing down of natural subsidence comes into one of these oscillating phases in which the depositing activity

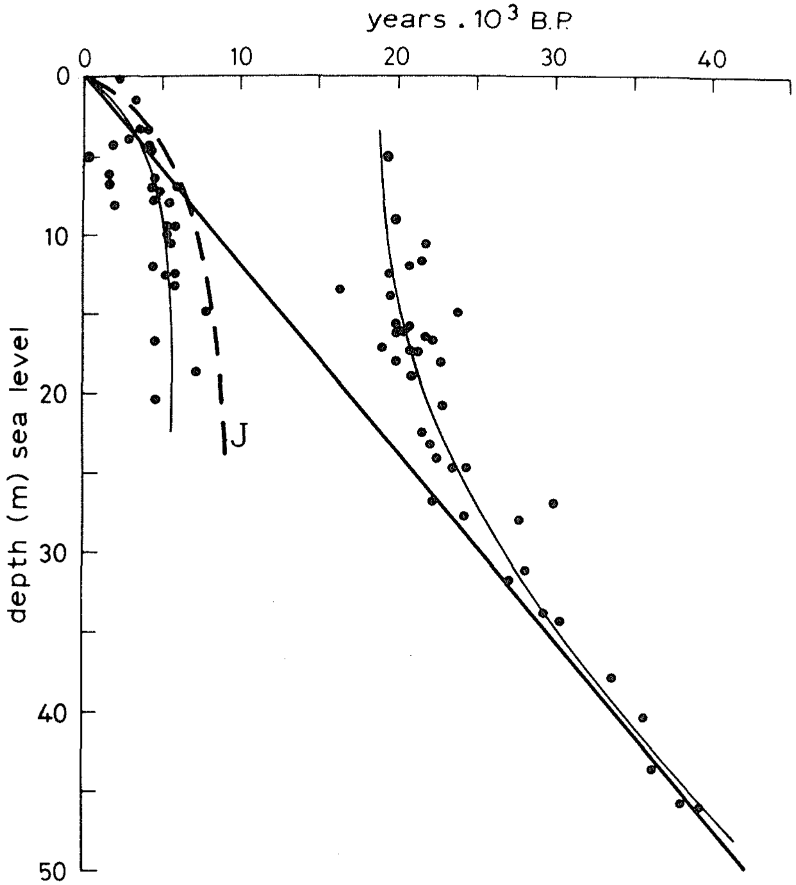

FIG. 3 - Relationship between age and depth (vs m.s.l.). Pleistocene samples appear in the right hand side cluster, while the Holocene ones on the left. The gap between them corresponds to the erosion period. For both clusters the interpolating curves are reported (light solid line). The heavy line is the linear common interpolation. The dashed curve (J) represents the eustatic rising of sea level after the last glacial period (Jelgersma, 1961).

seems reduced, even though we miss the extra-regional geological-climatic reasons for the moment. For the local events, it is our opinion that man's intervention on the basin to safeguard the lagoon environment, begun in 1400 with the diversion into the sea of the Piave and Brenta rivers to avoid the filling in of the lagoon, are notably involved in this quiescent phase. There has been a sudden drop in sedimentation due to the lack of solid fluvial yield and therefore there has been a reduction in the rate of natural subsidence, accompanied by environmental interactions already described by

Gatto and Carbognin (1981). This new equilibrium, a phase of relative environmental stability, still existing at the beginning of the century, has been deeply modified and upset by man's intervention, essentially tied to intensive exploitation of groundwater from the aquifer-aquitard system which in the last decades has caused an induced land sinking. The latter occurrence added to the natural one has contributed to worsen the Venetian problem.

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SUBSIDENCE IN THE HOUSTON-GALVESTON AREA OF TEXAS

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Abstract

The withdrawal of large quantities of groundwater in the Houston-Galveston area of Texas has resulted in declines in excess of 75 m in the potentiometric surface in the Chicot aquifer and 100 m in the Evangeline aquifer from 1943 to 1977. The water-level declines have caused subsidence in excess of 3.0 m to occur in the area.

The introduction of surface water to the low-lying areas along the coast and, beginning in 1975, regulation by the Harris-Galveston Coastal Subsidence District has resulted in decreases of groundwater pumpage in these areas from 9.4 cubic m/sec in 1976 to 4.1 cubic m/sec in 1982.

The District measures subsidence using borehole extensometers and precision leveling. This subsidence data and the information obtained from the use of three computer models allows the District to establish regulations necessary to control future subsidence.

Introduction

The Houston-Galveston area is located on the gently sloping coastal plain of Texas adjacent to the Gulf of Mexico (figure 1). Houston is the fourth largest city in the United States and the site of its third largest port. Galveston, located 65 km southeast of Houston, also contains a port and is a well-developed resort area. The Houston-Galveston area, which includes the world's largest petrochemical

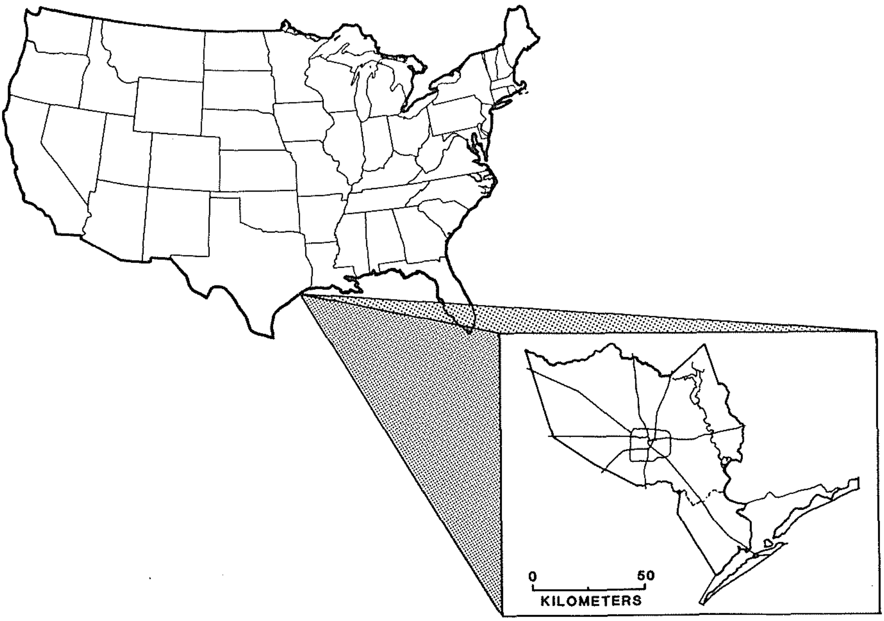


FIG. 1 Harris-Galveston Coastal Subsidence District.

complex, has experienced rapid economic growth. The withdrawal of vast amounts of groundwater for municipal, industrial and agricultural use has caused dramatic declines in artesian pressure, resulting in subsidence and accelerated fault movement.

Although subsidence has been occurring in the Houston-Galveston area since the turn of the century, it was not recognized as a significant problem until the 1940's. Successive levelings in the Texas City area uncovered discrepancies which indicated that the land surface had subsided.

Since that time, additional subsidence has been recorded throughout the area with the largest amount being 3.0 m at the Houston Ship Channel. In the Brownwood subdivision of Baytown, approximately 30 km east of Houston, over 2.5 m of subsidence has been measured. This area has been so devastated by subsidence-related flooding that most of the homes have been abandoned and the residents forced to relocate.

In addition to subsidence, a current study indicates that fault movement is increasing in areas which withdraw large amounts of groundwater (Holzer and Verbeek, 1983). With over 321 km of active or potentially active faults in the Houston-Galveston area, a significant amount of property is endangered by the additional fault activity.

Estimates indicate that over 4,451 square km within the District have experienced at least 0.3 m of subsidence since 1906. Land along the coast is particularly affected by loss of elevation. The coastal area's vulnerability to tidal inundation focused attention on the subsidence problem and brought citizens and elected officials together to seek a solution.

Harris-Galveston Coastal Subsidence District

In 1975, the Texas Legislature created the Harris-Galveston Coastal Subsidence District. Harris and Galveston Counties, which comprise the District, contain an area of 5,620 square km. Other adjacent counties are also affected by subsidence. The District has been charged with the responsibility "to provide for the regulation of the withdrawal of groundwater within the boundaries of the District for the purpose of ending subsidence which contributes to or precipitates flooding, inundation, or overflow of any area within the District, including without limitation rising water resulting from storms or hurricanes."

The District is governed by a Board of Directors composed of fifteen members, who are appointed and serve a two-year term. The Board conducts official District business at monthly meetings.

The mandate to control groundwater withdrawal is met by requiring permits for the drilling and operation of wells with an inside casing diameter greater than 12.7 cm. The quantity of groundwater that may be withdrawn is made part of the permit. Permits may also be restricted with other conditions. Public hearings are scheduled annually for each permit, wherein the District staff, well owners and all other interested citizens may comment on the amount of groundwater withdrawal to be permitted for a particular well. Based upon the testimony presented at the public hearing, a recommendation is made to the Board of Directors who authorizes the amount of groundwater withdrawal. In 1983, the District had 2,342 permitted wells which withdrew a total of 18.8 cubic m/sec.

The District has no taxing authority but collects a fee from each well owner issued a permit. The fee, which is used for the operation of the District, is based on authorized groundwater withdrawal. The rate of the fee is determined annually by the Board.

To insure an accurate accounting of the amount of groundwater withdrawn by a particular well, the District requires each well to have a meter. A member of the District staff makes a personal inspection of each well annually. If it appears that an individual has violated the terms or conditions of a permit or has failed to comply with the law, the District may file suit and may be entitled to recover monetary penalties.

Information and data collected through field inspections, pumpage reports, and permit applications are merged into a data base which includes physical characteristics of the well, withdrawal history, permit terms and conditions, location and ownership. This data base is used to generate permit renewal applications, permit fee statements, permits, various mailing lists and numerous analytical reports. The operation of the District is accomplished by a sixteen member staff.

Hydrogeology

The Houston-Galveston area is underlain by two producing aquifers (Chicot and Evangeline) and a confining layer (Burkeville). These artesian aquifers are comprised of alternating lenticular deposits of sand and clay, few of which are continuous areally. Although the system has been divided into two aquifers, they are hydraulically interconnected and are termed "leaky." In the absence of heavy pumpage, the water levels in both the Chicot and the Evangeline are similar.

Groundwater withdrawal from the aquifer system reduces the potentiometric surface and increases the intergranular pressure of the underground materials. Compression of the aquifer system results from the increase in intergranular pressure, however, the clay strata exhibit much more deformation or compaction than the surrounding sand layers.

The Houston-Galveston area has grown at a rapid rate since the 1940's. Groundwater withdrawals have increased proportionately and as a result, potentiometric surfaces have declined drastically. Figure 2 shows the changes in water levels from 1943 to 1977 in wells producing from the Evangeline aquifer. Maximum declines, in excess of 100 m, have occurred generally in the central to northwest area of Houston. Water level declines in wells producing from the Chicot aquifer show a similar pattern with a maximum in excess of 75 m.

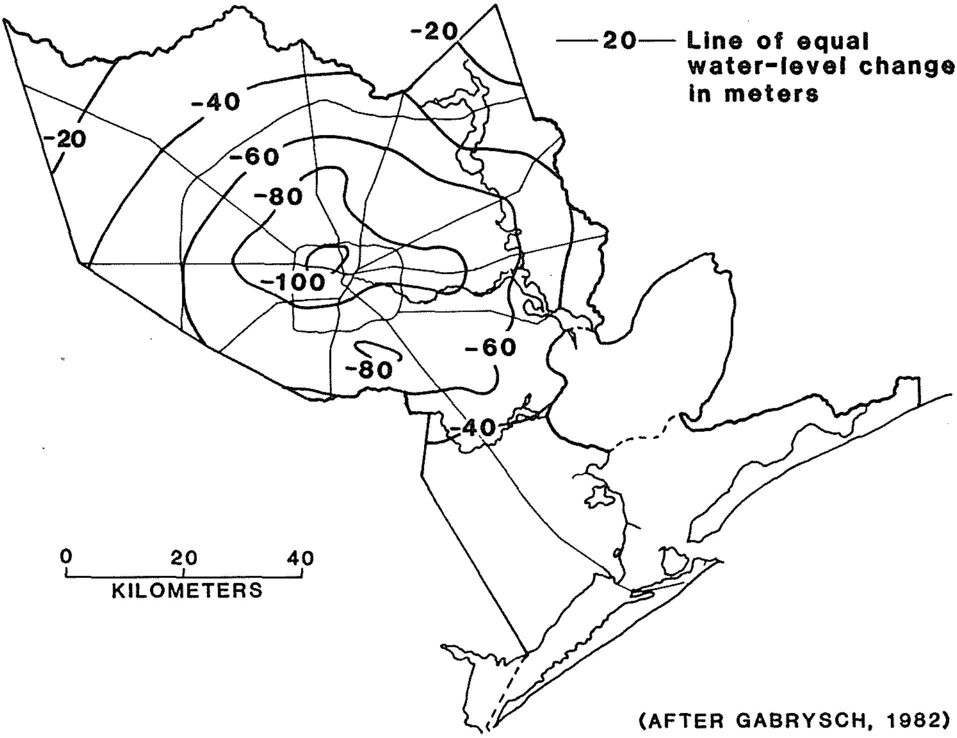
These declines in potentiometric surface have caused subsidence in excess of 3.0 m in the Houston Ship Channel area (figure 3) from 1906-1978. Another area of localized subsidence has occurred near Texas City.

The introduction of surface water to the coastal area has significantly decreased the rate of subsidence. In 1976, raw surface water began to be supplied to industries along the Houston Ship Channel. Use of surface water by these industries increased to 6.7 cubic m/sec by 1981. This surface water augmented the existing 4.4 cubic m/sec of treated surface water already being supplied to the coastal area by the City of Houston. Recently, 1.6 cubic m/sec of treatment capacity has been completed by the City of Houston and other municipalities to provide additional surface water to these low-lying areas. Groundwater withdrawals in this area decreased from 9.4 cubic m/sec in 1976 to 4.1 cubic m/sec in 1982 with the addition of surface water.

The potentiometric surface for the Evangeline aquifer in the Houston Ship Channel area increased as much as 43 m (figure 4) as a result of the reductions of groundwater pumpage from 1977 to 1982. The increase in the potentiometric surface for the Chicot aquifer for the same period was as much as 40 m. The potentiometric surface increase in the aquifers led to a decrease in the rate of subsidence in the area.

Subsidence Monitoring

The two primary methods by which the District measures subsidence are borehole extensometers and precision levelings. The future use of a satellite oriented global positioning system is being pursued.



(AFTER GABRYSCH, 1982)

FIG. 2 Approximate water-level declines in the Evangeline Aquifer, 1943-1977.

A borehole extensometer is a monitor well specifically designed to record changes in vertical displacement between the land surface and the bottom of the well. Thirteen extensometers ranging in depth from 130 m to 915 m have been installed. Seven of the extensometers are relatively shallow having been constructed to relate compaction to depth. Six of the extensometers have been constructed to depths below which artesian pressures are not declining and compaction is not occurring. Therefore, these extensometers monitor the total amount of subsidence and are precise enough to yield daily subsidence data.

Records from two of the extensometers designed to measure total subsidence are shown in figure 5. The record from the Pasadena monitor shows how the subsidence rate decreased in 1977 following the introduction of surface water in late 1976. Analysis of the record indicates no net loss in elevation although fluctuations due to seasonal changes in soil moisture of the upper clays have occurred. The record from the Addicks monitor may be used to show what is happening in an area which continues to rely on groundwater for its source of supply.

Since 1906, several leveling surveys have been conducted in the Houston-Galveston area to determine land-surface elevation. Historically, the surveys had to go as far as 110 km to reach a stable point of reference. It has been determined that three of the extensometers, designed to be total subsidence monitors, could be used as stable points of reference for the vertical adjustments being made in 1983. Three other extensometers, constructed at later dates, are expected to be used as control points during the next major leveling scheduled for 1986.

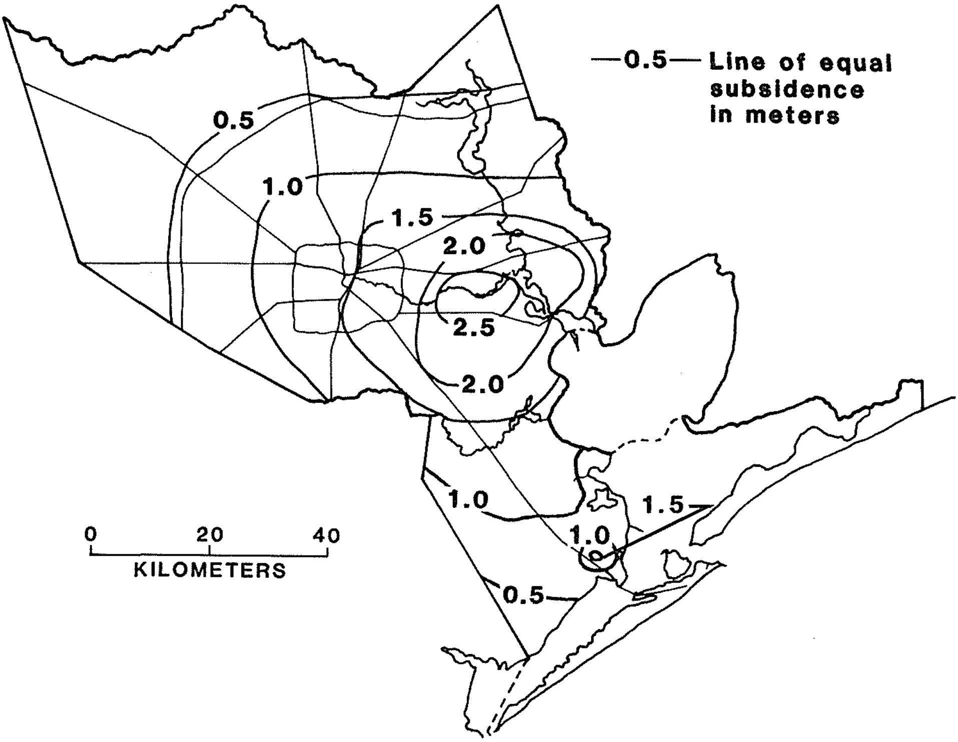


FIG. 3 Subsidence, 1906-1978.

The District is considering the use of a global positioning system (GPS) during the 1986 leveling program. GPS is a system of receiving stations at specified locations linked with a series of satellites. These satellites, using the known location of a point, will converge over a period of time on the relative location of a point or points, depending on the number of receiving stations. The system is useful for traversing long distances quickly with the use of satellites. The GPS is not expected to replace precision leveling but can be used to obtain subsidence data in areas removed from established leveling lines, or where extensometers are not present.

Computer Models

Due to projected growth in the Houston-Galveston area and the relatively high cost of surface water, it is important that the District's regulatory policy be developed to allow maximum utilization of the groundwater resource while preventing the damaging effects of subsidence. A computer modeling system was developed for predicting subsidence which would be caused by groundwater withdrawal in various locations and amounts.

The modeling system consists of three individual computer models (figure 6). The first of these models predicts the future need for water annually through the year 2020 by amount and specific location. The input of this model consists of water demand projections based on census tract data. In addition, various regulatory schemes which control the proportion of groundwater to surface water used in any part of the District can be input. The output from this model predicts groundwater withdrawals from each area of interest for each year

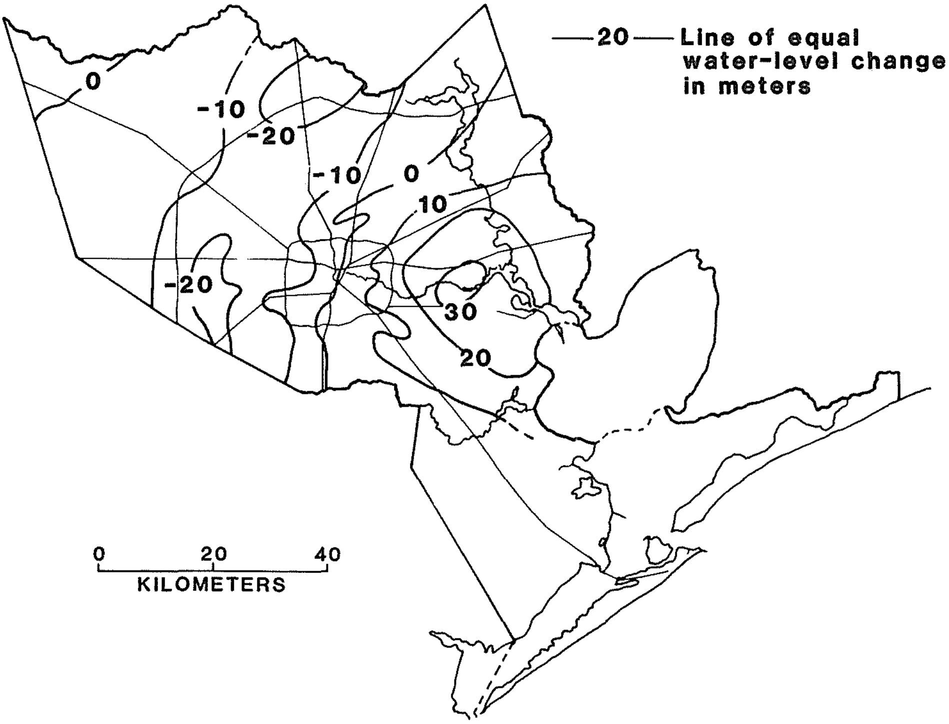


FIG. 4 Approximate water-level changes in the Evangeline Aquifer, 1977-1983.

between 1984 and 2020, and becomes input data for the groundwater model.

The groundwater model used is the one originally developed by Trescott (1975) and modified by Meyer and Carr (1979). The model is a three-dimensional, finite-difference computer code for solving transient flow equations in the aquifer system. Five vertical layers include each of the two major aquifers, the intervening confining beds and a surface layer supplying recharge to the system. The grid consists of 1,200 nodes, each representing an area of two and one-half minutes latitude by two and one-half minutes longitude or about 18.4 square km/node. Input to this model consists of all of the normal aquifer parameters such as transmissivity, leakance, and storage coefficient as well as starting values for the potentiometric surface and both historical and predicted pumpage for each of the nodes. This input data set consists of over 350,000 individual data items. The model's output is a series of plots showing changes in the potentiometric surface in each of the two major aquifers for the period from 1980 to 2020.

The third and final model of the series is a clay-compaction model originally developed by Helm (1975, 1976a, 1976b, 1978) and modified to utilize the potentiometric surface changes from the groundwater model as input. This compaction model is one-dimensional and solves the Terzaghi equations for consolidation. It has been calibrated at 21 selected sites in the District. Data for the calibration at each of these 21 sites was derived from geologic and geophysical well logs, releveling data, and potentiometric surface changes. The output from this model is a graph for each one of the sites showing projected subsidence through the year 2020.

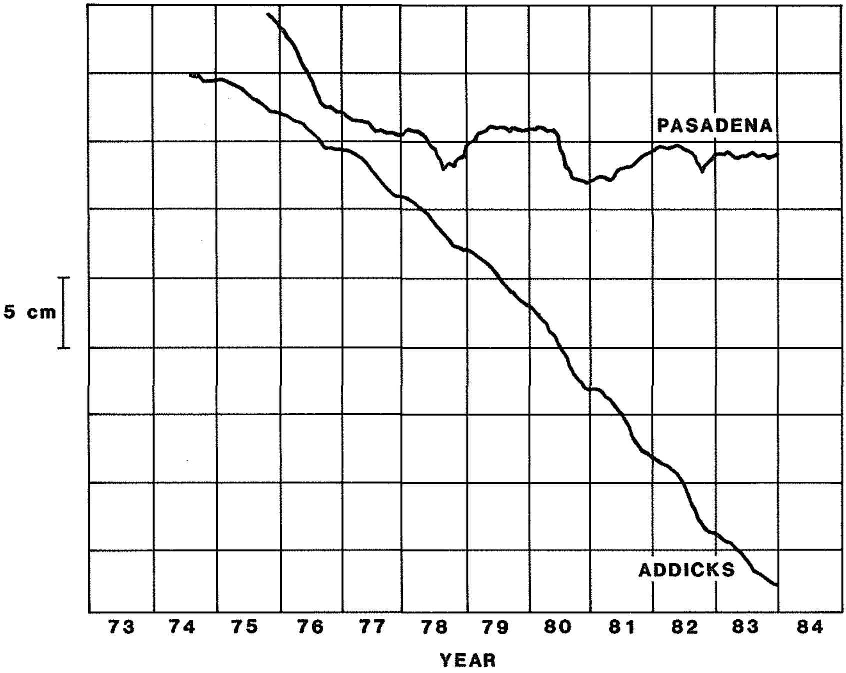


FIG. 5 Borehole extensometer data.

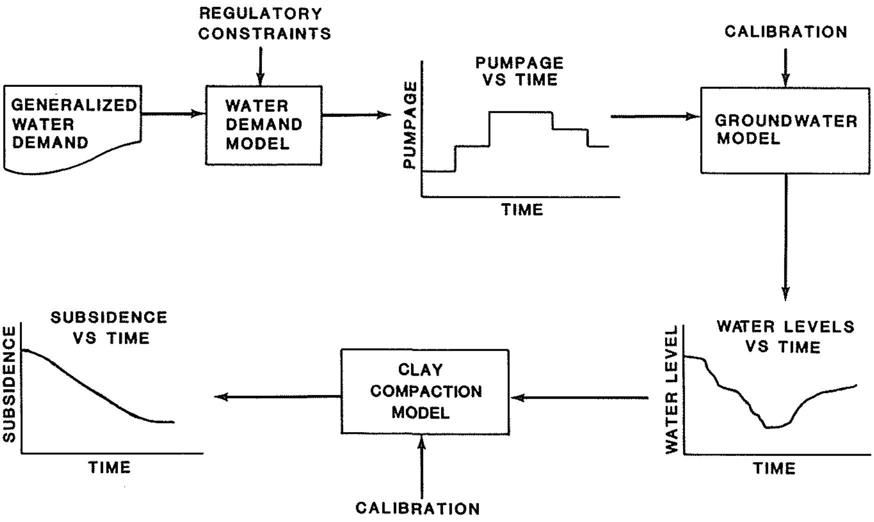


FIG. 6 Flowchart of modeling system.

The entire modeling system has been installed on the District's Hewlett-Packard model 1000 minicomputer. The largest of the models, the groundwater model, is run in $\frac{1}{2}$ megabyte of core and represents one of the earliest successful attempts in the United States to run a full-scale, three-dimensional groundwater model on a minicomputer.

Technology and Regulation

The use of the computer modeling system for projections of subsidence has already had significant impact on planning for the future. One of the first scenarios that was modeled considered that all additional water needs in the area would be met by groundwater withdrawals through the year 2020. The subsidence resulting from these new demands on the aquifers was clearly unacceptable and showed the need for a continuing program of groundwater withdrawal regulation.

The District is prohibited from selling or distributing water and, therefore, must rely on its regulatory authority. The use of current technology will provide the ability to develop long range regulatory plans. These long range plans will assist others in planning for surface water projects as well as provide additional information for flood control planning.

Conclusion

Groundwater is a valuable natural resource that has contributed to the dynamic development of the region. The low cost, high quality and abundant supply of groundwater led to an overuse of the aquifers. Subsidence began to cause serious economic problems. Groundwater regulation and conversion to surface water has reduced rates of subsidence in low lying coastal area. The use of new technology will aid in the development of regulatory plans which will provide the ability to manage damaging subsidence.

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ECONOMIC IMPACT OF SUBSIDENCE ON WATER CONVEYANCE IN CALIFORNIA'S SAN JOAQUIN VALLEY, U.S.A.

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Abstract

Agriculture in the arid San Joaquin Valley depends on irrigation. Depletion of aquifers prior to surface water delivery by the Central Valley Project (CVP) caused subsidence, locally up to 8-9 m. Subsidence-related rehabilitation of old, and design modification of new CVP canals cost $67 million. Surface water importation is achieved by pump lifting water from the Sacramento-San Joaquin Delta. The Delta was a tidal marsh with up to 16-m-thick peat deposits. Oxidation of peat in the reclaimed Delta caused locally over 6.5 m of subsidence which have resulted in numerous levee breaks and floods, multimillion dollar losses and repairs. The proposed $1 billion levee rehabilitation will neither arrest subsidence nor stop levee breaks due to hydrostatic head, poor foundations and high seismicity. Delta floods increase Delta salinity by intrusions of San Francisco Bay water and increased evaporation. Such increases will jeopardize multi-billion dollar agriculture in the valley.

Introduction

Land subsidence related to human activity is a continuously increasing geologic hazard (Bolt et al., 1975). Its economic impact has increased with the growing world population and technology (Prokopovich, 1972). It is not a coincidence that the world's largest continuous terrain affected by man-induced land subsidence is located in California (Ireland et al., 1982), the most populated state of the United States of America. Well-documented, past monetary impact of subsidence in California is large, but the potential future impact of subsidence here, as described in this paper, is not emphasized, if not overlooked, in the literature. The following paper is based on three decades of studies of subsidence by the U.S. Bureau of Reclamation. The ideas expressed in this paper are, however, those of the author and may not represent the official view of the Bureau.

Location and General Data

The state of California is the most southwestern state of the continental U.S.A. It is also the most populated state in the Union, with numerous sophisticated industrial complexes and major metropolitan areas including San Francisco, Los Angeles and San Diego. However, the No. 1 state industry is agriculture.

State landscapes range from "endless" flat valleys, to snow-covered mountain ranges and from desert to majestic forests of Giant Sequoia. The central part of the state is a major, deeply alluviated structural trough, some 600 km long and 70 km wide. It is located mostly between the Sierra Nevada on the east and the Coast Ranges on the west, and is known as the Central Valley of California. This valley consists of two interconnected valleys - the Sacramento Valley on the north and the San Joaquin Valley on the south (Fig. 1). Most of the precipitation occurs during the wet winter seasons at higher elevations in the mountains of northern California, while

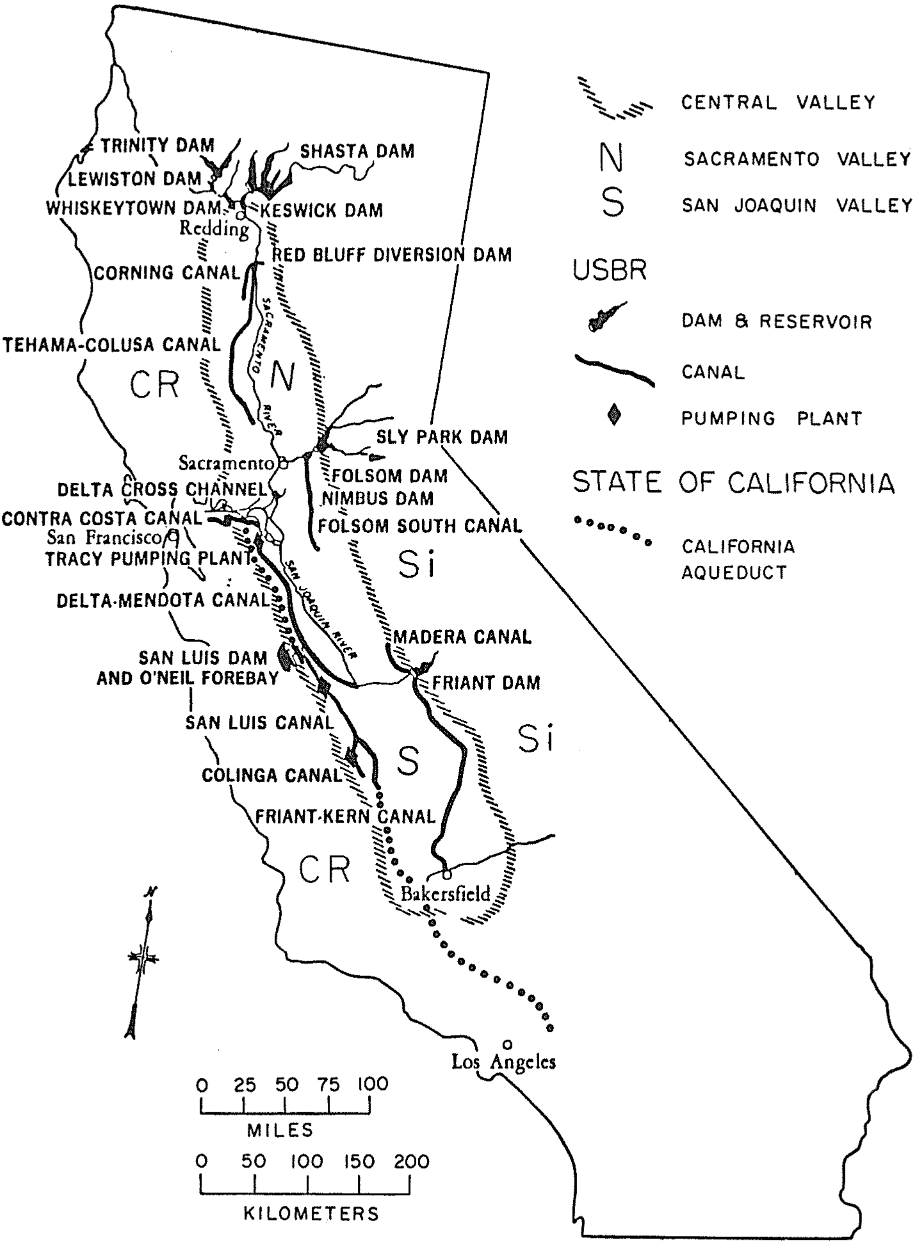

Fig. 1. Map of California showing major features of the Federal Central Valley Project and State California Aqueduct. Si= Sierra Nevada. CR= Coast Ranges.

the climate of the Central Valley and Southern California is dry, two-seasonal, semiarid to arid.

It may sound paradoxical, but warm, semiarid regions are particularly attractive to modern agriculture. Because of aridity, such regions have little or no damaging precipitation, while enjoying an abundance of sunshine, long growing seasons and frequently potentially fertile soil. The California San Joaquin Valley may serve as one of the best examples of ultra-modern agricultural development of such an area. The present agricultural production of the valley is on the order of 7.5 billion dollars and amounts to about half of the total agricultural production of the state. There are some 200 different crops grown in the valley of which 8 provide over 10 percent of total world production.

Modern agriculture in the valley is possible only with irrigation. As a general rule, the development of local ground-water resources is less expensive than construction of major conveyance systems, which requires construction of canals, dams, pumping plants, transmission lines and powerplants. Therefore, development of local ground-water resources usually precedes construction of complex conveyance systems.

Major importation of irrigation surface water into the valley was achieved in 1950-51, after completion of two initial Central Valley canals-the Delta-Mendota and Friant-Kern Canals. The Central Valley Project, or CVP (Anonymous, 1981), designed and constructed by the Federal Bureau of Reclamation, consists of several dams and associated hydropower plants and tunnels in the mountains of northern California (Fig. 1). The major project dams are Shasta, Keswick, Trinity, Lewiston, Folsom, and Friant Dams. Controlled releases of winter runoff and snowmelt stored behind these dams has eliminated much seasonal flooding in the valley. These releases are delivered by gravity flow into the Sacramento-San Joaquin Delta. From the Delta, the water is pump lifted into the 188-km-long Delta-Mendota Canal for further conveyance into the San Joaquin Valley. An essentially similar scheme is followed by the California Aqueduct of the State of California Water Project or SWP (Anonymous, 1974). The major source of State Water Project water is Oroville Reservoir in the Sierra Nevada. Both CVP and SWP water is pump lifted into the joint Federal-State San Luis Reservoir in the Coast Range Foothills, from which it is delivered to the southern half of the San Joaquin Valley (Anonymous, 1981) and (SWP only) Southern California (Fig. 1).

Subsidence in the San Joaquin Valley

Natural recharge of aquifers in arid regions is usually restricted, and major development of such aquifers results in their overdraft. Under geologic conditions existing in the San Joaquin Valley, overdraft (with piezometric decline locally exceeding 150 m) of aquifer systems resulted in spectacular subsidence, locally up to 8 - 9 m (Ireland, et al., 1982). At the present time, most of the valley floor is affected by variable amounts of subsidence, including 3 interconnected areas of major subsidence, known as Los Banos-Kettlemen City, Arvin-Maricopa, and Tulare-Wasco areas (Poland, et al., 1975). Maximum subsidence and maximum past piezometric decline occurred in the vicinity of San Luis Canal on the west side of the valley.

The existence of subsidence was not recognized during the design and early construction phases of the first CVP canals (the Delta-Mendota and Friant-Kern Canals). Construction and postconstruction subsidence interfered with operation and maintenance of these canals, and associated

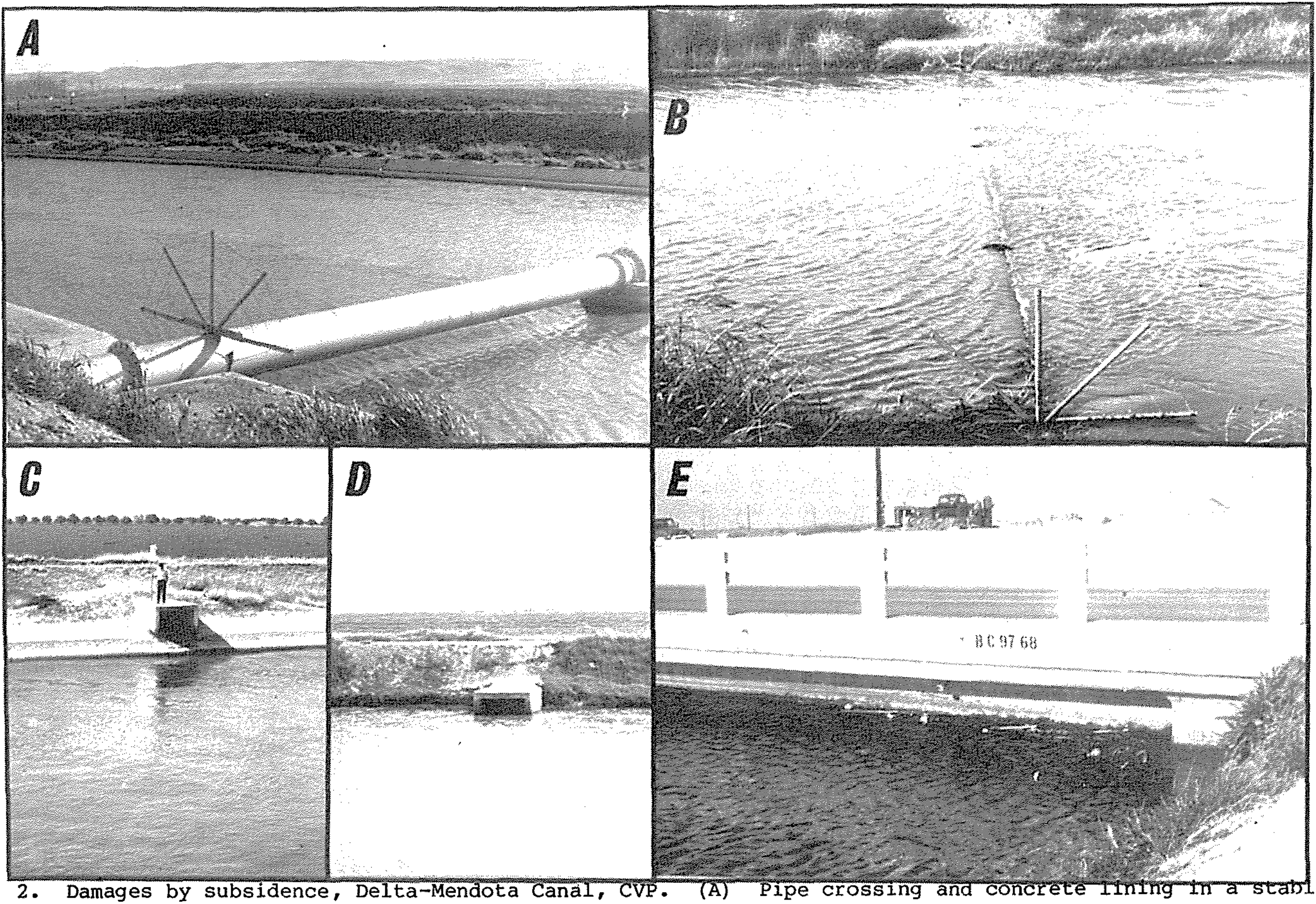

Fig. 2. Damages by subsidence, Delta-Mendota Canal, CVP. (A) Pipe crossing and concrete lining in a stable area (B) The same, in area affected by subsidence. Drain outlets in stable (C) and subsiding (D) areas. (E) Concrete bridge in subsiding area. Bridge bottom is submerged.

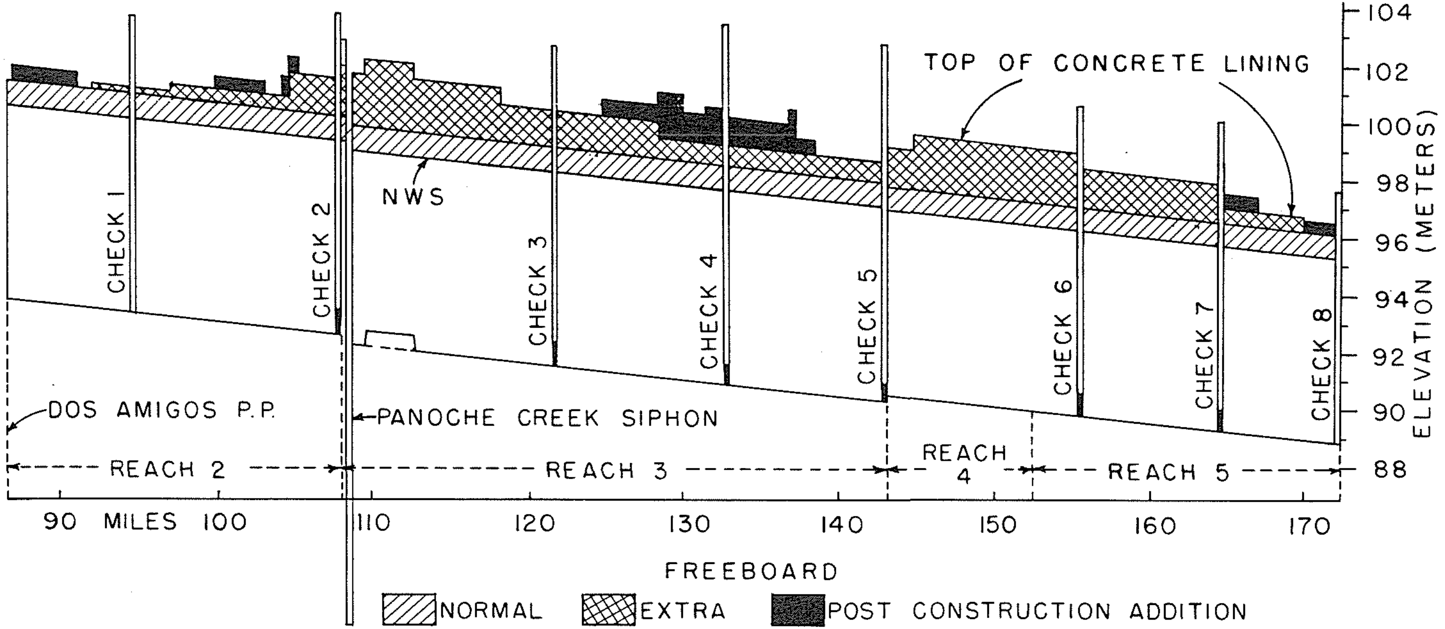

Fig. 3. San Luis Canal - Extra freeboard provided to compenstate for anticipated construction - post-construction subsidence.

structures (Fig. 2), and eventually required costly rehabilitation of several canal reaches (Prokopovich, 1969, 1984).

An attempt was made, therefore, to estimate amounts of future subsidence, and include some provisions for this subsidence into the design of three new canals in the valley - the San Luis and Coalinga Canals, and San Luis Interceptor Drain (Prokopovich, 1971, 1975)(Fig. 3).

In addition to this so-called regional subsidence caused by an overdraft of ground water, two reaches of San Luis Canal and some 188 km of associated distribution pipelines were affected by a peculiar form of subsidence known as hydrocompaction (Prokopovich, 1984A). Costly preconstruction wetting of selected alinements was conducted to prevent post-construction damages. Total cost of rehabilitation and design modification for all CVP canals during the last two decades amounted to $41 million (Prokopovich and Marriott, 1983), i.e., expressed in 1983 dollars, $67 million or $71 million, including hydrocompaction (Fig. 4).

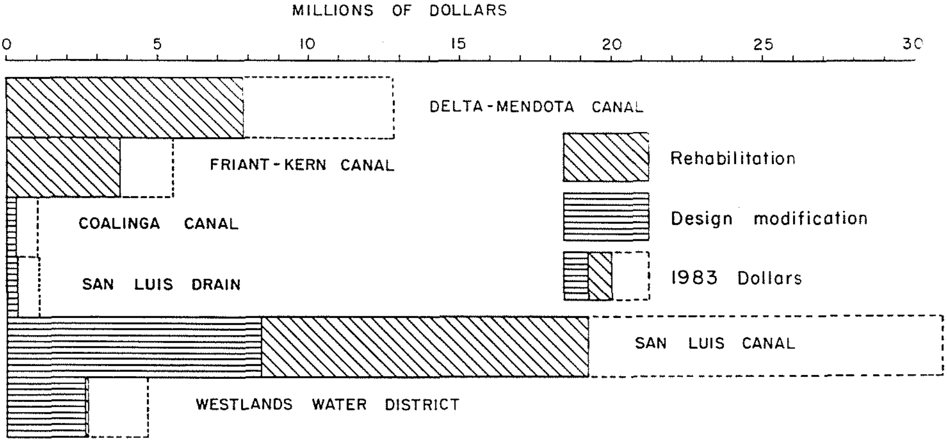

Fig. 4. Rehabilitation and Design Modification Expenditures, CVP, California.

Subsidence of Peat in the Sacramento-San Joaquin Delta

The so-called Sacramento-San Joaquin River Delta, or simply, the Delta (Fig. 5), is actually a Quaternary estuary filled with thick, mostly peaty, Holocene sediment (Shlemon and Begg, 1975). The maximum thickness of up to 12-16 m of peat occurs in the west-central section of the Delta. The area occupies a major structural basin with several nearby major active faults (Kearney, 1980). Some buried faults are present also within the Delta (Kearney, 1980; Shlemon and Begg, 1975). The Delta occupies some 2,800 km^2 of original freshwater tidal marsh and opens into the salty San Francisco Bay, connected with the Pacific Ocean. The area contains some 60 major islands separated by some 1,125-km-long net of deltaic channels and sloughs,surrounded by a net of manmade levees, some being over 100 years old. The over 167,000 km^2-Delta watershed encompasses about one-third of the state of California, and Delta water discharge amounts to one-half of all California riverflow (Kahrl, 1979). The original Delta landscape was flat with elevation close to sea level. The only exceptions were broad,

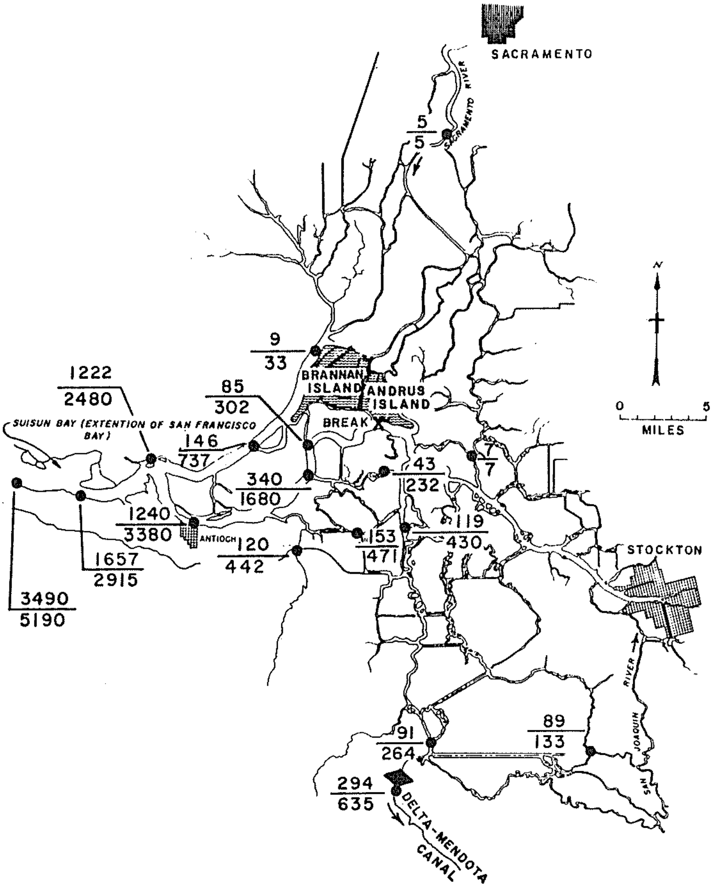

Fig. 5 Sacramento-San Joaquin Delta. Saline intrusion caused by 1972 flooding of subsided Brannan-Andrus Islands. Upper numbers indicate chloride content a day before the break; lower numbers indicate chloride content during the postbreak "salinity peak".

low, sandy-silty natural levees deposited by ancient floods and some eolian dunes. Present environmental conditions, including wildlife, are, however, entirely modified by agricultural developments and past gold mining in the Delta's watershed. Reclamation of the Delta, which started in the mid-1850's by an "improvement" of natural levees, requires both irrigation and drainage. Current annual value of Delta crops is about $375 million (Burke, 1980; Newmarch, 1981). The Delta has several developed gas fields, and is crossed by two navigational deep water channels, several important water and gas pipelines, a railroad and high voltage powerlines. The district is an important fishing and recreational area. Most important is the fact that the Delta is a key link in the giant Federal and State water conveyance system diverting northern California water into the San Joaquin Valley (and Southern California).

Cultivation and drainage of peaty deltaic islands changed the originally anaerobic soil condition into aerobic, resulting in an acceleration

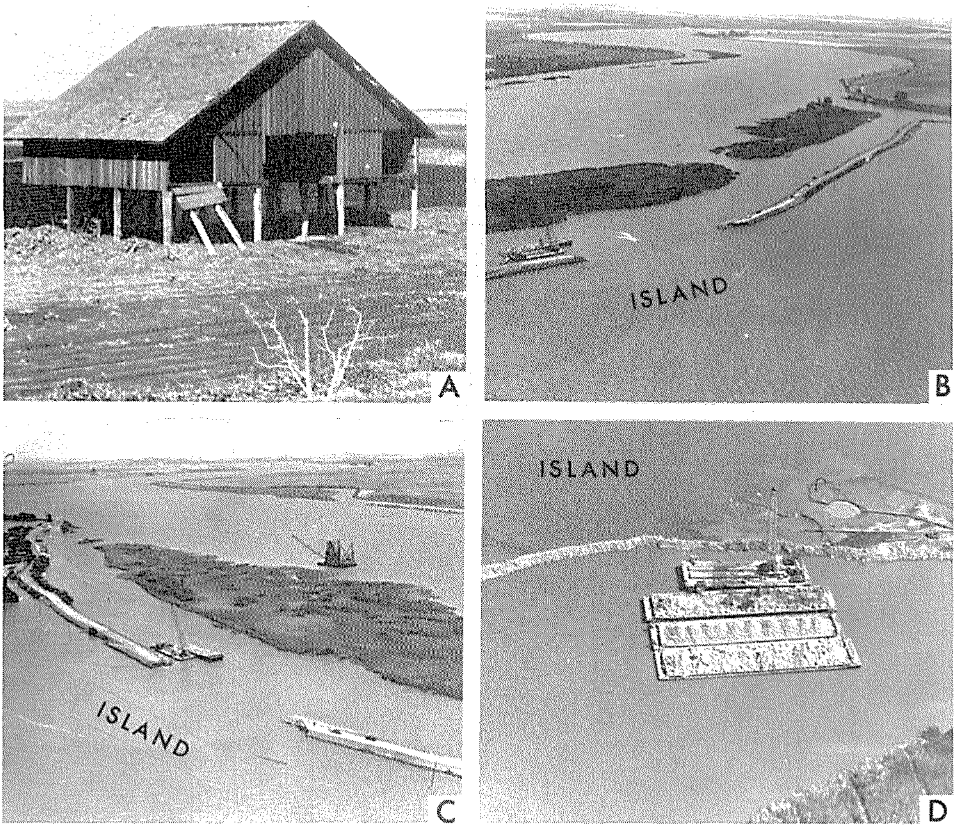

Fig. 6. Subsidence of Delta peat. A. Subsidence-elevated barn, built on piles, Holland Tract, 1959 (photo courtesy of California State Department of Water Resources). B., C., & D. June 19, 1979 flooding of Brannon-Andrus Islands (photo courtesy of U.S. Army Corps of Engineers).

of biochemical oxidation of peat in the warm California climate. Land subsidence (Fig. 6) generated by such oxidation was additionally aided by shrinkage, compaction, wind erosion, burning, possible tectonic movements and other processes (Newmarch, 1981). Present subsidence rates vary from 7.6 cm/yr in the western Delta to traces on its eastern margin. Due to subsidence, the surface of most of the Delta islands is below sea level and islands have became "bowl shaped" depressions with much terrain located below the water level of surrounding channels. The maximum subsidence, in an order of 6.5 m, occurs in the west-central section of the Delta. Present Delta islands are protected from flooding only by a net of frequently poorly designed and constructed levees, some of which are 100 years old.

Conclusions

The future economic impact of subsidence far exceeds the $67 - 71 million used to-date by the Federal Bureau of Reclamation for rehabilitation of old and design modifications of new CVP canals in the San Joaquin Valley. Water quality in the Delta, which determines water quality of CVP and SWP conveyances to the San Joaquin Valley, is controlled by: (1) inflow from the feeding rivers, (2) tidal intrusions of salty water, (3) evaporation within the Delta, (4) water consumption within the Delta, (5) water export by CVP and SWP, and (6) local pollution. Prior to the construction of CVP dams, the Delta experienced severe increasing salt intrusions which caused some crop damages and salinization of Delta soils. Controlled water releases during dry seasons, after construction of CVP (and SWP) dams, eliminated such intrusions. Past levee breaks and flooding in the Delta, particularly during dry seasons, resulted in a notable increase in salinity of Delta water (Fig. 5). Eventual similar but major salinization of the present Delta, which serves as the major source of CVP water, seems to be unavoidable due to continual subsidence of reclaimed peaty deltaic sediments and related collapses of levees surrounding subsided Deltaic islands, and flooding of islands. Immediate increase in salinity by "soaking" of Bay water into the Delta due to flooding will be accelerated by gradual increase of salinity due to evaporation from flooded islands. Particularly dangerous is the possibility of a major salt intrusion due to a major earthquake which can cause a rapid collapse of numerous levees.

Such salinity increases could eventually jeopardize water quality of future conveyances and harm the immense agriculture of the San Joaquin Valley.

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LAND SUBSIDENCE IN THE AREA OF MODENA, PO VALLEY, NORTHERN ITALY

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Abstract

The artificial land subsidence phenomena which occurred in the town of Modena (Italy) during the past decades are described, emphasizing the geological and hydrogeological conditions of the subsoil. The subsidence is clearly provoked by lowering of the piezometric surface, estimated in about 10 m. This lowering causes variations of the pore and effective pressure values in the silty and clayey levels present in the first 120 m of depth, corresponding, from a hydrogeological point of view, to a *multicompartimental monostratum* aquifer. Land subsidence reached maximum absolute lowering values little inferior to 1 m, fully justified by the geotechnical characteristics of the subsoil. The differential lowerings of the ground have induced remarkable lesions in the various buildings, due to the underground anisotropy. Moreover, some anomalies of the groundwater chemistry (F and Zn ions) have been recorded and are perhaps connected with the clays'interstitial water squeezing.

Introduction

The township of Modena is placed in the Apenninic margin of the Po Valley (northern Italy), at a level of about 35 m above sea level and at a distance of 15 Km from the first elevations of the Apennines range and of 140 Km from the Adriatic sea coast-line. During the past decades, the urban area and its outskirts have been interested by remarkable artificial subsidence phenomenon, with absolute ground lowering which reaches maximum values of about 1 m, causing severe damage to many monuments and buildings of the historical town centre. This situation was put into evidence when some geodetic levelling surveys were carried out between 1974 and 1977, and when, in the following years, some papers were published describing the static conditions and restoration methodology of the Dukes'Palace (XVIIth Century). This is one of the most important monumental buildings interested by lesions caused by differential lowerings of the foundation ground (Cestelli Guidi, 1978; Croci, 1980; Righi, 1980; Martinotti et Alii, 1981)

In the 1980,the Municipality of Modena started an organic study of the phenomenon, planning and setting out a new geodetic levelling network, connected with a bench mark situated in a geologically stable area of the Apennines (Russo, 1984). At the present, with the sponsorship of the Municipality of Modena, specific geotechnical surveys of the town centre underground are being executed. These surveys consists of Dutch

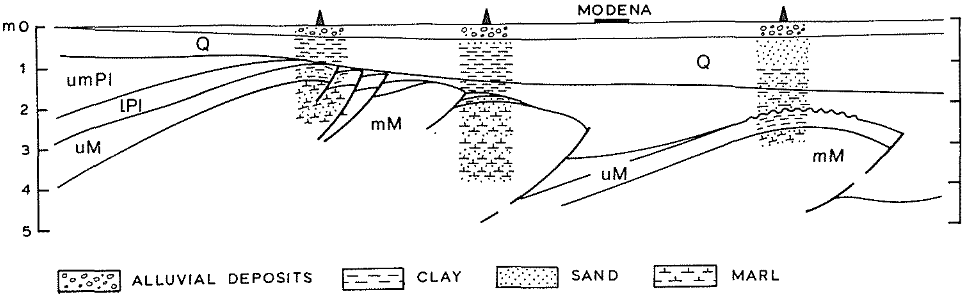

Fig. 1 - Deep geological conditions beneath the town of Modena (Pieri and Groppi, 1981) u, m M: upper, middle Miocene; 1, um, Pl: lower upper-middle pliocene; Q: marine Quaternary.

friction-cone penetrometer tests, geotechnical boring and laying out of Casagrande piezometers and strain meters, aimed at the individuation of the ground thickness subject to subsidence induced strains, and to a better definition of the town subsoil geotechnical characteristics. About these parametres only a few papers have been published up to now (Cancelli, 1984; Cancelli and Pellegrini, 1984). At the present, instead, there is a good knowledge of the hydrogeological characteristics of the town, both for the geometrical and physical structure of the aquifer (Colombetti et Alii, 1980; Pellegrini and Zavatti, 1980) and for the nappe flow field and its evolution in space and time (Various Authors, 1981-a).

Geological and hydrogeological setting

Information regarding the geological characteristics of the underground of Modena derives mainly from geophisical logs and from boring lithostratigraphies performed for the searches of hydrocarbons and ground waters. Beneath the town of Modena, the substratum formations at the base of the alluvial cover are met at an average depth of 250 m, 2-3 Km South of the town centre and 350 m from the ground-level, northwards. The bedrock is made up of a clay sequences with rare conglomerate levels in a transition environment, and of marine clays. The marine formation top corresponds to upper-middle Pleistocene, while the total thickness of the Plio-Quaternary marine formations (overconsolidated clays and weakly cemented sands) is found between 2,000 and 3,000 m of depth, respectively South and North of the town. Fig. 1 shows the deep geological conditions beneath the town of Modena (Pieri and Groppi, 1981). The alluvial cover (upper-middle Pleistocene-Holocene) is constitued by alternations of gravels with a sandy or silty-sandy matrix and by silty-clayey levels. Fig. 2 shows the structural conditions of the alluvial cover of the high Modena plain (Colombetti at Alii, 1980); in the first 180 m of underground

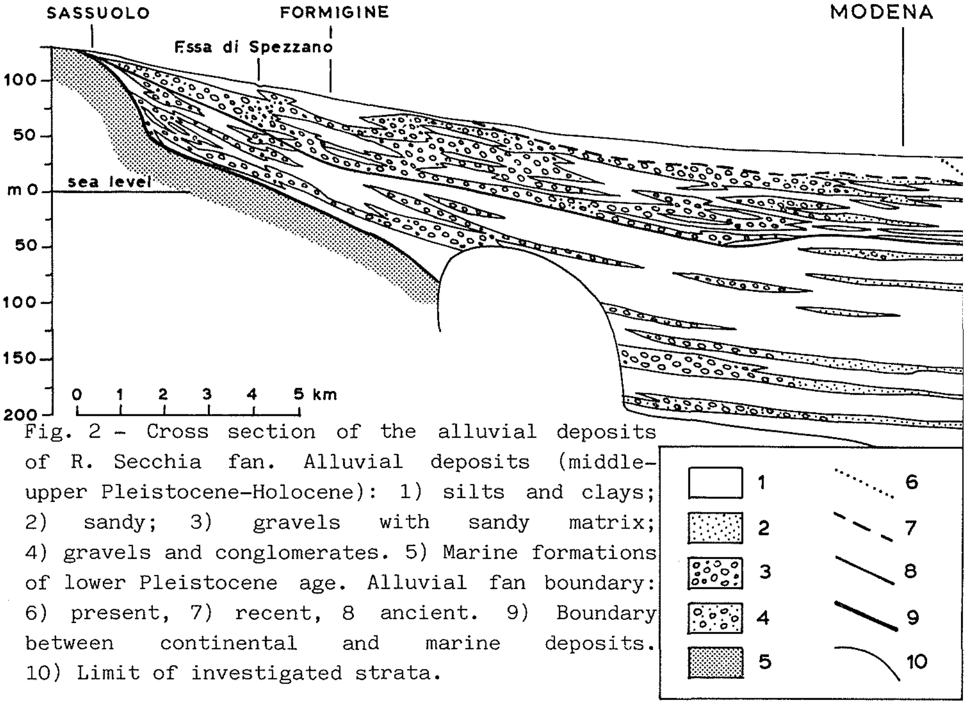

Fig. 2 - Cross section of the alluvial deposits of R. Secchia fan. Alluvial deposits (middle-upper Pleistocene-Holocene): 1) silts and clays; 2) sandy; 3) gravels with sandy matrix; 4) gravels and conglomerates. 5) Marine formations of lower Pleistocene age. Alluvial fan boundary: 6) present, 7) recent, 8 ancient. 9) Boundary between continental and marine deposits. 10) Limit of investigated strata.

the gravelly levels have a percent thickness ranging from 20 to 40% (Various Authors, 1981-b). This thickness usually decreases from S to N, turning away from the Apenninic margin.

The lithostratigraphic logs of some deep wells, drilled S and SW of Modena, have showed that beyond 180-200 m and up to the top of marine formations (300 m circa), the gravelly levels have reduced frequence and thickness, less than 5% of the considered interval. The top of the first gravelly levels, always around Modena, is found at a depth between 17 m in the southern part of the town, and 35 m beyond in the northern one.

The alluvial deposits, on which the town of Modena was built, belong to the alluvial fan of the River Secchia and to those of other minor water courses. The alluvial fan of the R. Secchia extends for about 70 Km, corresponding, from a hydrogeological point of view, to a sector of the widespread Po Valley aquifer system. This sector, although affected by the recharge of the R. Secchia, is not defined by boundary conditions with no-flows or with space and time univocal flows. The boundary conditions of the R. Secchia alluvial fan are as follows: the hill margin corresponds to a no-flows boundary (impermeable); the fan apex to a fixed flow, with recharge from the river to the aquifer. Up to a depth of 120 m, measured near Modena, but varying and generally increasing from S to N according to

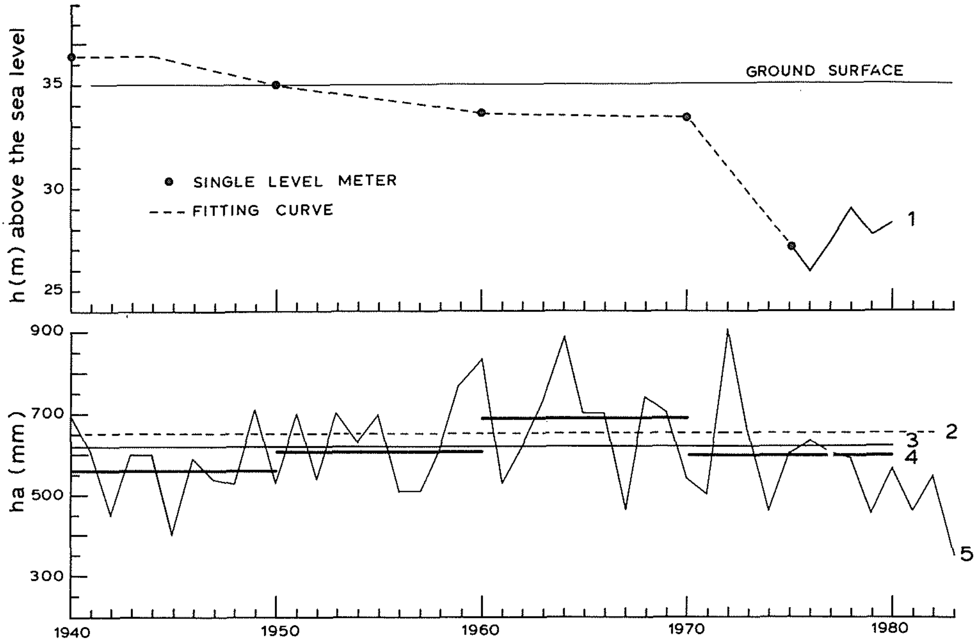

Fig. 3 - Evolution of groundwater piezometric level (h) in Modena (top part) from 1940 to 1983, compared with rainfalls (ha): 1) piezometric level; 2) rainfall average value from 1850; 3) idem, for 1940-1983; 4) idem, for 1940-1980 decades; 5) idem, annual values for 1940-1983.

the depositional model of the fan itself, the alluvial deposits from a multicompartimental monostratum aquifer with unconfined waters in the high sector of the plain, with confined waters near and N of Modena due to the presence of thick impermeable covering layers (Pellegrini and Zavatti, 1980).

At the present, the groundwater of the R. Secchia fan tends to a deficit. The inflows are represented by underdrainage from the water-course (0.5 m ^{3}/s) in the upper plain, by infiltration of rainfalls and irriguous waters (from canals), and they have been estimated at about 3 m ^{3}/s. The inventory deficit is caused by many factors: first, the groundwater overdraft, at least equal to inflows, is estimated just for the Municipality of Modena at about $42.8*10^6$ m^3/year, half of which is for domestic consumption (Various Authors, 1981-a). Secondly, the inflows have decreased during the last 30 years, because of the urbanization of widespread permeable areas in the high plain, which reduced by 20% the rainfall infiltration areas available. Moreover the excavation of gravels from the river bed has canalized its shape, decreasing the underdrainage toward the aquifer. The variation of the grounwaters inventary and of its factors is shown by the development trend of the potentiometric surface in the Modena area.

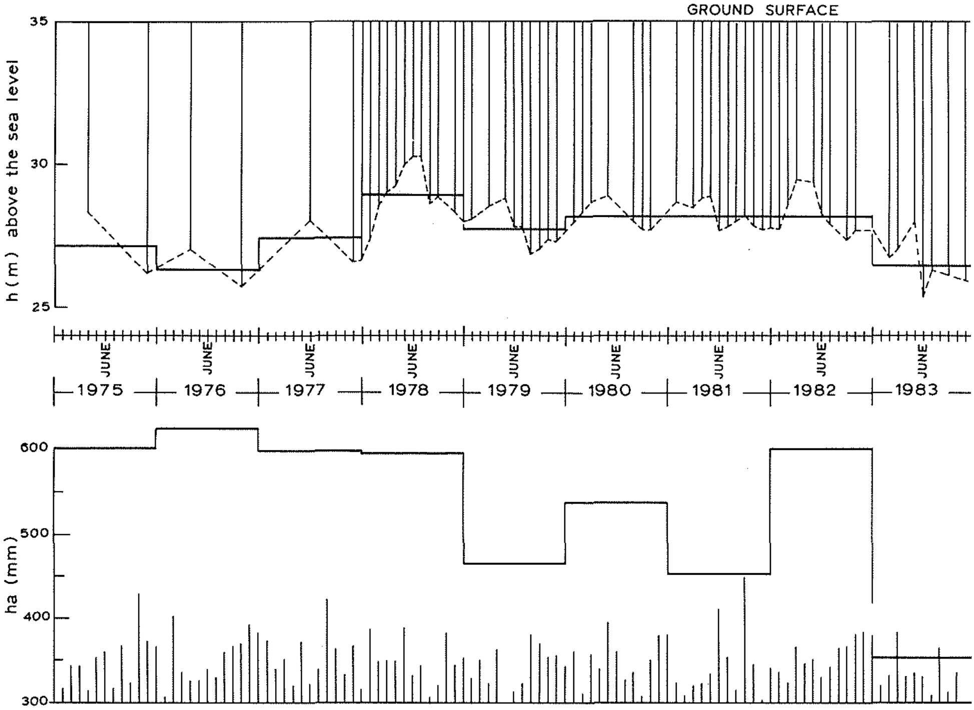

Fig. 4 - Evolution of groundwater piezometric level h (top part) from 1975 to 1983 in Modena, compared with rainfalls ha (bottom part): detailed analysis.

The water level decreased from 1.80 m above ground-level, during the pre-war years, to the ground-level between 1945-1955, to -1.50 m in 1960, and up to -9 m average value of 1983 (Fig. 3).However during 1975-1983, cyclic intervals were recorded with development trend of opposite sign; the extreme values ranged between -12 m in July 1976 and - 3 m in June 1978.

During the years 1982-83, characterized like 1979, by a negative trend, the low total flow rates of the R. Secchia and the precipitations, definitely below average (about 50% of previous values: Fig. 4), played a major role. In the years 1975-76 such a correlation between rainfall and piezometric levels does not take place, because of the pumpages which had the most important part in the groundwater level lowering. In the past two years the marked groundwater lowering, caused by a reduction of the rainfall inflows, was relatively limited due to a decrease of the overdraft, since the introduction of water recycling plants in some of the main water-consuming industries of the town. Fig. 5 shows in detail the groundwater potentials in the area of Modena during 1983, considering the water level in 55 wells. The potentiometric surface appears conditioned by

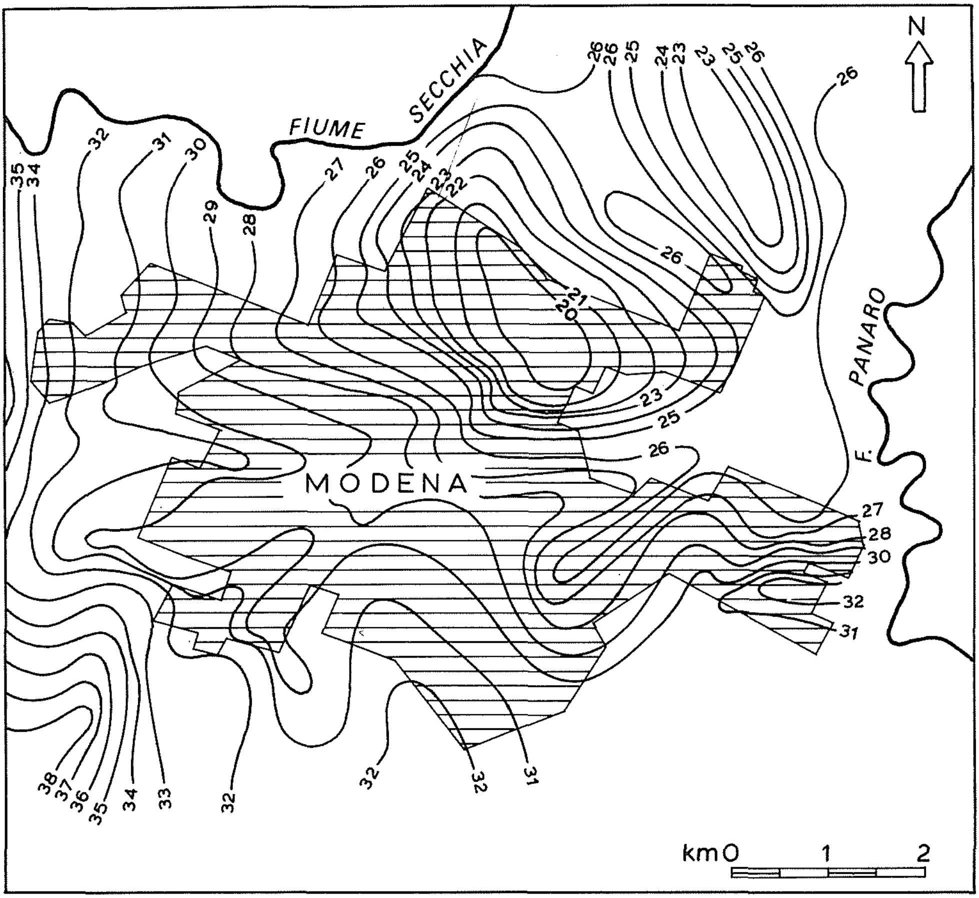

Fig. 5 - Detailed survey of groundwater potentiometric surface in Modena, May 1982 (m above sea level).

presence of important pumping stations, giving rise to drawdown in the piezometric levels.

The geotechnical characteristics of the subsoil have been illustated by Cancelli (1984) and by Cancelli and Pellegrini (1984) and can be summarized as follows. In the first part of the subsoil, usually investigated during foundation ground tests (up to 25 - 40 m), the prevailing lithotype is made up of more or less silty clays, in great part inorganic. The degree of consistency of these materials is widely varying: the undrained Cu cohesion ranges between 20 and 100 KPa, with the highest values concentred mainly in the first 10-15 m from the ground level (this being an index of overconsolidation due to nappe fluctuations and surface drying). As for the shear resistence in drained conditions, the ϕ angle generally varies between 20° and 24° ; c' cohesion often differs from zero, conforming to the degree of overconsolidation of part of the deposit

(Cancelli, 1984). Some geotechnical investigations carried out in the North outskirt of the town, have shown that the first 10 m of clays are overconsolidated, with an overconsolidation ratio OCR generally ranging from 2 to 4. Below a depth of 10 m the soil seem to be normally consolidated, but also slightly overconsolidated lenses are present at random (OCR≤2) Considering the alluvial deposition environment,the most likely cause of overconsolidation might be found in repeated periods of exposition to the air, afterwards covered by new alluvial deposits (Cancelli and Pellegrini, 1984); at deeper levels the natural aging of the deposits should also be considered. In the urban area, in particular between 10 and 15 m in depth, some samples are definitely underconsolidated; this may be related to the marked lowering of the water table in recent decades, thus explaining the present phenomenon of increased land subsidence. Values of maximum consolidation pressure inferior to the present geostatic pressure were obtained also for the deep clay layer underlying the Dukes'Palaces, so that the subsidence phenomenon involves at least in part also deeper layers, and isn't limited only to the first clayey level. The presence of underconsolidated layers is not found in the North sector of the town, where the lowering of the piezometric level has been up to now much more contained (within a few meters).

Land subsidence

The town of Modena is located in the southern sector of the Po Valley, which is a large sedimentary basin, where during the Pliocene and even more in the Quaternary a differentiated subsidence began, which was very marked in nearly all the plain, reaching one of the highest worldwide values: a few kilometers North of Modena the Pliocene base is at a depth of over 6,000 m. Referring to the average formations thickness values recorded near Modena, the following values indicating the subsidence rate may be obtained:

0.4 mm/year in the Pliocene (5.4 M - 1.8 M years b.p.);
1.2 mm/year in the lower-middle Pleistocene (1.8 M - 0.8 M years b.p.);
0.37 mm/year in the middle-upper Pleistocene (0.8 M years b.p.);
3 mm/year in the last 2,000 years, up to 1945;
40-80 mm/year in the last 20 years.

The value for the last 2,000 years was obtained considering that the Roman archaeological level is buried under 6 m of alluvial deposits and presuming that absolute topographic altitudes are substantially unvaried due to compensation by alluvial deposits. The geodetic levelling surveys carried out between 1972 and 1983 and the relative data elaborations (Various Authors, 1981-a; Russo, 1984) showed that near the town of Modena the land subsidence rate starting from the pre-1970 years, has increased at lest tenfold compared with the last 2,000 years circa. The most pronounced lowering between 1950 and 1979 took place in the town centre with maximum values of 83.7 cm. Although the evolution of this phenomenon in time is not yet fully understood, the average annual subsidence may be calculated at 4 to 8 cm/year in the areas of greatest intensity.

The subsidence, highly differentiated in the town centre in relation to the ground anisotropy, is in general more marked North of the centre, where the water pumpages for industrial consumption are concentrated and where the silty-clay sequences prevail (over 80% in the first 120 m in depth) over the coarser ones. The differential lowerings have naturally caused lesions in numerous public and private buildings in the town centre, some of which are important monuments. Also a tract of the R. Secchia dyke on the NW outskirt of the town had to be raised by 70 cm, a height corresponding more or less to the soil lowering. As for the causes of differential subsidence, registered around Modena, in recent years, like other towns in the Po Valley, there is no doubt about their being artificial, considering the anomalous lowering rates compared with historical or geological ones. There is in fact a clear connection with the lowering of the water-table recorded on both a regional (Various Authors, 1981-a) and a local level (Martinotti et Alii, 1981). This connection is particularly evident comparing the soil lowering-isolines (Russo, 1984) and the potenziometric map surveyed in Modena in May 1982 (Fig. 5).

Groundwater chemical alterations

The chemistry of the groundwaters of the R. Secchia alluvial fan is influenced by the feeding conditions of the river. The waters have a high saline content (Chlorides, Sulphates and Bicarbonates) and a marked hardness (Calcium), up to over 50 French hydrothimetric degrees. The groundwater temperature varies between 10° and 13 ° C, increasing slightly up to a depth of 200 m altogether, in conformity with all the Po Plain basin, except a few areas, which is characterized by extremely low geothermal gradients. In the last few decades an increase in the temporary hardness of the water-supply wells of the Municipality of Modena, due to an increase of Bicarbonates. This increase is difficult to explain for now: it may be due to the increase of bacterial activity in the farm land of the permeable feeding areas with free CO_2 following organic pollution; or to acid rains caused by air pollution; or lastly to variations of the geochemical conditions underground, resulting from a high rate of exploitation of the aquifer. This last hypothesis could be linked to the anomalous presence of Fluorine and Zinc ions in the waters; however there are no available reference data for the past. In groundwaters the Fluorine isocone lines reproduce the geometry of the gravelly alluvial fans, with values up to 0.1 mg/l, signifying that these ion concentrations are inversely proportional to be thickness of the gravelly levels of the aquifer. North of Modena values up to 1 mg/l are reached, corresponding to thin sandy-gravelly aquifer levels, interbedded between silty-clayey layers. If on the one hand polluting sources can be excluded, on the other there is a clear connection with the underground lithostratimetry and with the exploitation rate of the aquifer.

Analogous conclusions may be drawned examining the area distribution of the Zinc concentrations in the groundwater, and also on the base of a detailed study carried out on the hydrochemistry of the waters from a well South of Modena always referred to the Zinc ion. The well, which drains different levels of the aquifer system, contains waters with larger concentration of Zinc ion, corresponding to the aquifer levels subject to a most intense pumping in the surronding area. The anomalous concentration of these ions, usually present in limited quantities and of a little significance for hydrogeological characterisation, could be determined by the lowering of the piezometric surface and by a subsequent variation of tension in the clays. This phenomenon could be considered concurrent with artificial subsidence: consolidation of the clays and sqeezing of the interstitial waters (Various Authors, 1981-a).

Conclusions

In the township of Modena natural land subsidence has gradually increased from the Pliocene up to the present day, with rate values that from the initial 0.3 mm/year have reached 3 mm/year in the last 2,000 years (Fig. 6).

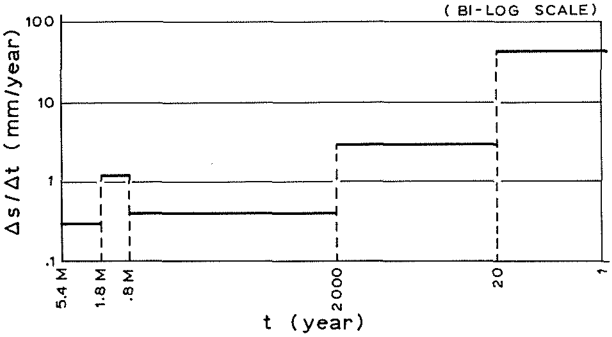

Fig. 6 - Rate values of subsidence from Pliocene to the present in the area of Modena.

Subsidence in the post-war period, with a rate of a fiew cm/year, may obviously be attribuited to artificial causes. Moreover a clear connection has been shown between land subsidence and the evolution of the piezometric surface which in Modena has undergone a lowering of about 10 m.

As for geotechnical interpretation of this phenomenon, the data available at present are too few, especially because of the complete lack of data relating to deep layers; therefore the following considerations necessarily rely on a large-scale extrapolation. The first 120 m of soil subject to an increase in tensional state can be referred to; this thickness corresponds to the aquiferous layer ("multicompartimental monostratum") undergoing pumpage. Only 80 of these 120 m will be considered, excluding the first 10 m already consolidated and 30 which may be attributed to gravelly layers. According to Cancelli and Pellegrini

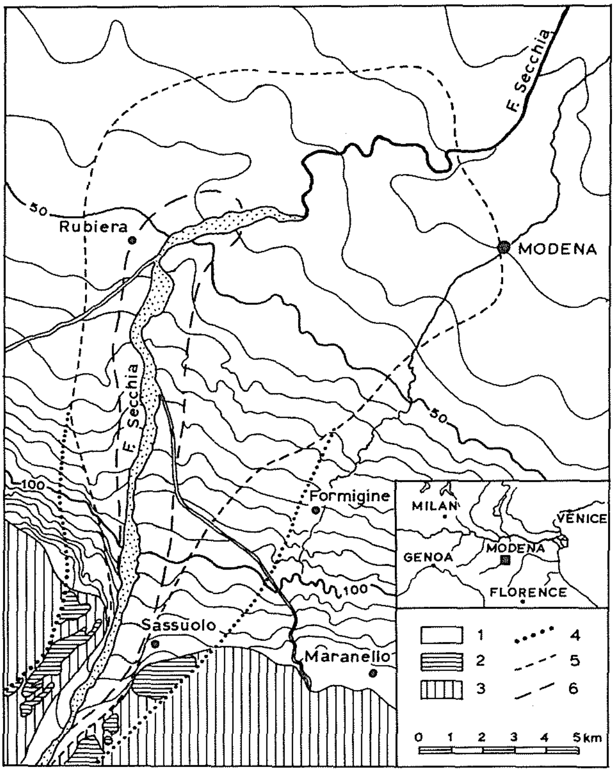

Fig. 7 - Alluvial fan of the R. Secchia: 1) alluvial deposits (with contour lines); 2) terraced alluvial deposits; 3) impermeable marine formations of the Apenninic margin. Fan boundaries: 4) ancient, 5) recent, 6) present.

(1984), assuming that the average total pressure variations is constant in time and in depth with Δp = 100KPa, and attributing to compressible soil a primary compressibility of Rc = 0.2 (without considering aging), a general oedometric packing would be obtained equal to 1.2 m. The absolute maximum lowering values, reported by Russo (1984) which are less than 1 m, give plausibility to this hypothesis. The present tendency of land subsidence in Modena should be towards the attenuations of the phenomenon, provided that the water table is kept at constant values in the future. The result of the geodetic levelling carried out in 1983 on the control network, which recorded maximum differential lowerings of 25 mm/year, increasable up to 30 considering also geological subsidence (compared with 40-80 mm of the previous years), could confirm this evolutive trend.

In Modena, the land subsidence seems to be accompanied also by anomalies in the groundwater composition, at least for what concerns the presence of some ions usually present in small quantities in the alluvial deposits groundwares, such as Zinc and Fluorine, among the considered parameters, that are influenced by the squeezing effect of the clay interstitial waters. Were this hypothesis verified, a long-term substantial alteration of the waters chemistry could be reached, with possible consequences for the use of groundwaters hydric resources (drinkable waters). The decision adopted by Municipal Authorities of Modena tends to privilege the use of surface waters, at least for industrial use; this is consistent with a policy for the rational use of water resources aiming to oppose land subsidence, since adequate norms are still lacking in Italy.

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LEGAL PERSPECTIVES ON SUBSIDENCE CAUSED BY GROUNDWATER WITHDRAWAL IN TEXAS, CALIFORNIA, AND ARIZONA, U.S.A.

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Abstract

Damages from subsidence caused by ground-water withdrawal include shoreline submergence, well-casing failures, and changes in gradients of canals, sewers, irrigation ditches, and streams. Potential damages from subsidence-related earth fissuring include structural damage to roads, railroads, and buildings, ruptured sewers and pipelines, and aquifer contamination. The ground-water doctrine of English rule of capture has prevented recovery for damages due to subsidence in Texas. In the landmark Friendswood case of 1978, the Texas Supreme Court added negligence as a cause of action for recovery. Texas also established the Harris-Galveston Coastal Subsidence District to control subsidence. In California, some water districts control subsidence with conservation measures, pump taxes, and importation of surface water. The 1980 Arizona Ground Water Management Act has minor provisions for controlling subsidence. The act seeks safe yield in Active Management Areas by the year 2025 using five management periods. The Subsidence Monitoring Plan for Arizona provides for a central data facility on subsidence in the State.

Introduction

Natural subsidence has caused problems for thousands of years. When geologic processes cause lake, bay, and ocean shorelines to sink below the water level, valuable property can be lost or damaged. Sinkholes that develop in karst terrains are dramatic examples of loss of subjacent support due to natural geologic processes. In addition, induced subsidence from the extraction of subsurface resources such as oil and gas, water, sulphur, salt, coal, and geothermal energy has become an economic and social problem. Among areas affected by subsidence caused by ground-water withdrawal are Mexico City, Mexico; Venice, Italy; Houston-Galveston, Texas; San Joaquin Valley, California; and Eloy Basin, Arizona.

Poland (1969) used soil-consolidation theory and the principle of effective stress (Terzaghi, 1925) to account for aquifer compaction and subsidence. In a confined aquifer, the fluid pressure provides some support for the overlying sediments. Decrease in pressure in the aquifer due to pumping causes an increased stress on the matrix of the aquifer. This stress results in compaction of the aquifer as a whole. In an unconfined aquifer, the water exerts a buoyant force on the matrix. As the water table declines, buoyancy decreases in the dewatered zone, increasing the effective stress throughout the unconfined aquifer. Fine-grained units of high porosity and low permeability usually are included in or are boundaries for confined and unconfined aquifers. When the pore pressure in the aquifer decreases relative to that in the fine-grained units, water flows from the fine-grained units into the adjacent coarser units until the pore pressures equilibrate. This process commonly takes from a few weeks to many decades, is mostly irreversible although it can be stopped by aquifer repressurization, and can account for most of the total subsidence of a system.

Earth fissuring occurs in alluvial basins in Arizona, California, and Nevada and is commonly attributed to differential subsidence due to ground-water-level declines over buried geologic structures (Peterson, 1962; Schumann and Poland 1969; Laney and others, 1978; Jachens and Holzer, 1979). In Arizona, nearly all fissures occur in alluvium less than 300 m thick near mountain fronts in such areas as Picacho, Casa Grande, Chandler Heights, Apache Junction, and Avra Valley. Fissures commonly open from a few millimeters to about 30 mm and range in length up to several hundred meters (Boling and Carpenter, 1978; Carpenter and Boling, 1980). No strike-slip movement has been observed, and dip-slip movement has been observed at only a few fissures. Fissures commonly erode into fissure gullies (Laney and others, 1978) of a meter or more in width and as much as 15 m in depth. One fissure system near Picacho, Arizona, is 15 km long and has 0.3 to 0.6 m of vertical offset in many places. At one U.S. Geological Survey study site on that fissure, horizontal opening of the fissure was 15 mm, and vertical offset was about 21 mm from June 1980 to June 1983.

In some places in alluvial basins in Arizona, it is difficult to distinguish subsidence and fissuring caused by natural aquifer drainage from induced subsidence and fissuring and to determine which pumping caused which subsidence or fissuring. The first known fissure occurred in 1927 near the town of Picacho prior to major pumping and ground-water-level decline in the basin (Leonard, 1929). Some fissures are associated with natural dessication above the water table (Neal and others, 1968). In addition, some alluvial basins have indications of preconsolidation stress (Holzer, 1981; M. C. Carpenter, unpublished data, 1984) which may be due in part to natural ground-water drainage followed by recovery in the geologic past prior to major pumping in the basins. Although certain measurements of horizontal strain, fissure movement, and subsidence can be correlated with pumping cycles of particular wells, several factors limit the ability to assign physical responsibility for damages. Among these are the commonly elastic or recoverable nature of short-term effects due to pumping of a single well or small well field; short-term noise effects on measurements of temperature, barometric pressure, and earth tides; long-term effects of regional drainage, fluctuations in recharge, and seasonal water-level decline and recovery; and various hydrologic effects such as boundaries and heterogeneities.

Several types of damage occur due to subsidence and fissuring. Damages from subsidence itself are generally due to submergence, as when shore lines sink in Houston-Galveston, Texas; Wilmington, California; Venice, Italy; and Tokyo, Japan. In Arizona, damages from subsidence itself include well-casing failures, and erosion and deposition due to changes in base levels of streams. Differential subsidence causes damage by changing or reversing gradients of canals, irrigation ditches, sewers, and streams. Costly measures have been taken in northeast Phoenix to minimize accumulation of explosive gas and maintain flow in sewers (Harmon, 1982). Fissuring increases the potential damage; and when vertical offset or shear occurs with or without fissuring, potential damage increases significantly. Buildings, petroleum-product pipelines, sewers, roads, and railroads can be damaged. In the Houston-Galveston area in Texas, localized differential subsidence without fissuring along preexisting fault planes has damaged houses, roads, and an airport (Holzer, 1976, 1984; Neighbors and Thompson, 1984). If pipelines and sewers rupture due to fissuring, aquifers can be quickly, and perhaps irreparably, contaminated from the direct recharge of petroleum products, sewage, etc. Cracks in remote areas may invite unlawful and dangerous dumping of hazardous wastes resulting in aquifer contamination. In the Tucson Basin, an area along the buried Santa Cruz fault has potential for differential subsidence and earth fissuring if water-level declines continue (Davidson, 1973; Anderson and others, 1982). This area coincides with a plume of trichloroethylene contamination in the ground water and vadose zone (Kreamer, 1983). If a well field is established to withdraw contaminated ground water in the vicinity of the fault, the consequent ground-water-level decline may cause differential subsidence and fissuring. If fissuring or incipient fissuring occurs in this area, the contamination could spread more rapidly in the vadose zone.

Scope

Several seemingly disparate topics merge to form the legal aspects of subsidence caused by ground-water withdrawal. These include the various forms of physical damages, the ground-water doctrines that apply in different jurisdictions, the laws of support, tort laws, restatements of torts, case law, and statutues. The sections on tort laws and the restatements of torts constitute background for the section on the Friendswood case that illustrates the conflicts of ground-water doctrine, laws of support, and torts. Additional sections present statutes that deal with subsidence as well as court tests of these statutes.

Ground-Water Doctrine

Ground water in the United States is generally governed by the English rule of capture, the American rule of reasonable use, correlative-rights doctrine, or appropriative-rights doctrine (Tank, 1983). The English rule, observed in Texas and many Midwestern and Eastern states derives from the English case of Acton v. Blundell, (1843) in which a quarry owner was sued by a neighbor because dewatering the quarry dried up the neighbor's well. The court held that a landowner has the right to absolute ownership of all the water he can capture which percolates under his land. The injury was termed Damnum absque injuria or injury without damage, indicating that there is no remedy under the law for the injury or damage suffered. Injury sustained can be a dry well, greater lift costs, or even subsidence as in Finley v. Teeter Stone, Inc. (1968) in Maryland. Pumping can be for the sale of water, irrigation, mine or quarry dewatering, etc. The strength of this doctrine is exemplified by courts granting summary judgments and refusing to try cases brought before them for subsidence damages (Friendswood Development Company v. Smith Southwest Industries, Inc., 1978).

The American rule of reasonable use holds in Arizona and several Midwestern and Eastern states. It is a modification of the English percolating-water rule that provides for a case-by-case determination of the reasonable use of water. Wanton waste and malicious use are excluded from reasonable use, as are transport and sale of water in some jurisdictions. Large losses through unlined canals, irrigation tail water, and inefficient irrigation practices, however, are usually considered to be reasonable use.

California adheres to the correlative-rights doctrine, which is an application of riparian law to ground water (Steelhammer and Garland, 1970). Landowners over a common aquifer are considered joint tenants entitled to a proportionate share of the ground water for beneficial use on overlying lands. Excess water not needed by overlying landowners can be appropriated for use elsewhere. In times of shortage, overlying landowners take reduced fair and proportionate shares. In addition, lowering ground water levels is considered unreasonable and can be enjoined even in the absence of injury (City of Pasadena v. City of Alhambra, 1949).

Most western States including New Mexico adhere to the doctrine of prior appropriation for ground water. This doctrine holds "first in time, first in right." The system deals with water shortages on the basis of seniority and is commonly administered by the State Engineer (Tank, 1983).

Avulsive and Non-Avulsive Subsidence

In coastal areas or along shorelines of lakes, subsidence may cause property lines to change if the lines are determined by water boundaries. If the water boundary changes rapidly enough to be perceptible, the change is called avulsive. If the change is imperceptible, it is non-avulsive. If land is gained or new land is created, it is termed accretion. If land is lost, it is lost by erosion or submergence. Submergence occurs by subsidence if the land sinks or by inundation if the water level rises. If a water boundary changes as a result of avulsive subsidence, there is no corresponding change in property ownership to coincide with the new water boundary unless the prior owner fails to reclaim the land. In the case of a non-avulsive subsidence, ownership of the submerged land goes to the State, Federal, or municipal government that owns the body of water; but, except in Texas, the land is determinable in favor of the prior owner if he restores it in a resonable time (Davis, 1976).

Laws of Support

Damages caused by subsidence can be recovered through the civil courts under common law dealing with lateral or subjacent support (Casner, 1977). In general, one who causes a loss of lateral support is absolutely liable, even in the absence of negligence or malice, for damages to the land in an unimproved state. In Texas, an owner has the right to lateral support, injunctive relief against its loss, or an award of damages, irrespective of negligence (Steelhammer and Garland, 1970).

Doctrines regarding loss of subjacent support are more complicated than those dealing with lateral support. In some jurisdictions, the common law of reasonable use or correlative rights applies, and the circumstances of each case are weighed. In Texas, there is no recovery if the removed resource was water but there can be recovery if the removed resource was coal. Further, in some cases involving mineral resources, one who removed subjacent support was held absolutely liable for damages to the land in its natural state. In other cases involving mineral resources, there was no recovery by the surface owner where subsidence was a natural and inevitable result of mining by the only known commercial method (Steelhammer and Garland, 1970).

Recovery for damages to structures is obtained under negligence in civil cases. One factor affecting recovery is whether the weight of the structure contributed to the failure of the land surface. The issue of recovery for damages to structures is further clouded by the fact that, in some cases, builders have at their own expense had to provide shoring of adjacent land-owners' buildings. Some ordinances and statutes exonerate a builder if he notifies his neighbor and is not negligent (Casner, 1977). Other statutes allow recovery depending on the depth of the excavation.

Some case law regarding subsidence comes from ground-water law, and recovery depends on the jurisdiction. If the English percolating-water rule applies, there is no compensation if the produced water is clear. If the water is muddy or contains sand, there may be compensation because solid material is withdrawn (Steelhammer and Garland, 1970).

Tort Laws

A tort is a wrongful act, in the absence of a contract, that results in damage or injury. The major torts are nuisance, trespass, and negligence. A nuisance is a wrong resulting from unreasonable, unwarrantable, or unlawful use by a person of his own property. A nuisance may be private or public depending on whether it affects a definite or indefinite number of persons in a community. Nuisance generally requires an unlawful act but in some cases nuisance *per accidens*, nuisance in fact, is recognized in the absence of an unlawful act. A court will consider the act itself, the place, and the circumstances, balancing equities between property owners each asserting his rights to use his own land, in determining whether a given act constitutes a nuisance. The defense that an individual purchased property with the knowledge of the existence of a nuisance, termed coming to a nuisance, is not usually sufficient to prevent abatement or recovery of damages (Sullivan, 1976).

Trespass is an invasion of another's rights as the result of an unlawful act. Trespass may be to personal property, to the person, or to realty. Trespass to realty includes injury to or interferences with another's possessions below ground (Sullivan, 1976). Trespass may be invoked along with subjacent support in subsidence cases.

Negligence is the failure to observe or perform a legal duty owed another that results in injury to the other. The act may be one of either omission or commission. For a defendant to be held liable under negligence, his act must be the proximate cause of the injury. In addition, the defendant must be shown to have departed from reasonable and prudent behavior under the circumstances. The doctrine of *res ipsa loquitor*, the thing speaks for itself, applies when the plaintiff can produce sufficient facts to warrant an inference of negligence. For this, the plaintiff must show that the instrumentality which caused the injury was under the exclusive control of the defendant and that, under ordinary circumstances, no injury would have occurred if the defendant had used proper care. Defenses against negligence include contributory negligence on the part of the plaintiff and assumption of risk (Sullivan, 1976). Commonly, subsidence cases are brought under nuisance and negligence.

Restatements of Torts

The American Law Institute is an advisory organization that published a Restatement of Torts in 1939 and a Restatement (Second) of Torts in 1969, each of which included positions on subsidence. Each of these restatements contained recommendations for wording and intent of court decisions and new statutes. Although the restatements are not themselves law, they have influenced court decisions and the development of statutes. Furthermore, the institute's position on subsidence has reversed. In 1939 the institute stated: "To the extent that a person is not liable for withdrawing subterranean waters from the land of another, he is not liable for a subsidence of the other's land which is caused by the withdrawal." In 1969, the Institute recommended instead: "One who is privileged to withdraw subterranean water, oil, minerals or other substances from under the land of another is not for that reason privileged to cause a subsidence of the other's land by such withdrawal." (American Law Institute, 1939, 1969). The Institute's changed position is significant in that it suggests the legal duty to refrain from causing subsidence.

Friendswood Case

In Friendswood Development Co. v. Smith-Southwest Industries, Inc., (1978), Smith-Southwest Industries and others along the west shore of Galveston Bay brought a class action in 1973 against Friendswood Development Co. and its owner Exxon Corp., alleging that defendants' pumping for the sale of ground water caused severe subsidence of plaintiffs' lands. Friendswood joined as third party defendants other parties pumping ground water in the area. The suit was brought under laws of support, negligence, and nuisance in fact. Plaintiffs alleged that defendants' wells were too closely spaced and too near the common boundary between plaintiffs' and defendants' lands. They also alleged that defendants pumped excessive quantities with foreknowledge from engineering reports that such pumping would cause subsidence and flooding of plaintiffs' lands. Defendants contended that subsidence was a problem in the area before they started pumping and that pumping from other wells in the area contributed to the subsidence. The trial court granted a summary judgment for the defendants, citing Acton v. Blundell (1843) and the doctrine Damnum absque injuria. The appellate court reversed and remanded, holding that a cause of action existed in nuisance and negligence. The Texas Supreme Court reversed, affirming the summary judgment of the trial court (Friendswood, 1978; Teutsch, 1979).

In its holding, the Texas Supreme Court applied the law to this case as it was at the time of the suit and as it was from 1964 to 1971 when most of Friendswood's wells were drilled. The court cited the Restatement (Second) of Torts (American Law Institute, 1969) and a subsidence amendment in the General and Special Laws of the State of Texas (1973) as too late to affect this case. The court held, however, that henceforth negligence would be added to malice and willful waste as a cause of action. Regarding the legislation creating Underground Water Conservation Districts and the Harris-Galveston Coastal Subsidence District, the court stated: "Providing policy and regulatory procedures in this field is a legislative function. It is well that the Legislature has assumed its proper role, because our courts are not equipped to regulate ground water uses and subsidence on a suit-by-suit basis," (Friendswood, 1978).

California and Texas Legislation and District Management

Several authors have indicated the need for effective statutes to control or prevent subsidence (Compton, 1961; Davis, 1976; Singer, 1976; Morris, 1980; and Kopper and Findlayson, 1981). It is more significant that courts have expressed the need as well (Houston and Texas Central Railroad v. East, 1904; Finley v. Teeter Stone, Inc., 1968; Friendswood 1978).

Approaches to subsidence management in California include a subsidence statute, water districts, and a governor's commission. The Anti-Subsidence Act of 1958, which was designed to maximize production of oil and gas fields, provides for a cause of action only when there is a direct violation of an order or decision of the state oil and gas supervisor (Statutes-California, 1958). The act provides for liability for withdrawal of subjacent support only in case of negligence. It does, however, provide a penalty of $1,000 per day per act of violation

(Compton, 1961). The Orange County Water District levies a pump tax on all ground-water pumping, imports water for uses including artificial recharge, and requires periodic reports of ground-water production. San Gabriel Valley uses a charge on pumping more than an adjudicated share of safe yield, has a recharge program, and controls ground-water storage in the basin. The Governor's Commission to Review California's Water Rights Law has recommended legislation that would give a State board the power to select an appropriate local agency to manage a ground-water basin or to form a ground-water management district. The management authority could regulate storage and use, buy, sell, and exchange water rights, and limit pumping in response to such adverse effects as long-term overdraft and subsidence. Farming interests have hindered the legislation because they perceive it as fostering bureaucracy and leading to more expensive water (Kopper and Findlayson, 1981).

Texas has used a combination of minor subsidence provisions in the Texas Water Code with a special act creating a subsidence district in an attempt to bring subsidence under control (General Laws-Texas, 1973, 1975). The Texas Legislature, in 1949, provided for the creation of Underground Water Conservation Districts (UWCD's) for "...conservation, preservation, protection, recharging, and prevention of waste of underground water...," while confirming private ownership of ground water (General Laws-Texas, 1949). This chapter was amended "To prevent drawdown, and to control subsidence and to prevent waste, 'the district may provide for the spacing of water wells and may regulate the production of wells,'" (General Laws-Texas, 1949, p. 560; Tyler, 1976). Although they have the authority, none of eight UWCD's has ever regulated pumping (Teutsch, 1979). In 1975, the Texas Legislature passed the Harris-Galveston Coastal Subsidence District Act that empowered the District to space wells, regulate pumping, meter wells, require annual reports of pumping, and collect permit fees not to exceed "'110 percent of the highest rate charged by the city of Houston for surface water to its customers,'" (General Laws-Texas 1975; Teutsch, 1979). The District is governed by a 15-member board appointed by mayors and commissioner's courts in the 2 counties. Agricultural and industrial interests are each assured at least one representative on the board. Stiff penalties and injunctive relief are provided for violations of the act or rules of the District. Agricultural interests were able to exclude an ad valorem tax from the act, decreasing its effectiveness in internalizing costs and providing incentives for switching to surface water. In addition, a subsequent ammendment limited the permit fee for agricultural users to a maximum of 70 percent of that for other users (General Laws-Texas 1977; Teutsch, 1979). The District has used its power to reduce pumping and has required some cities and industries to switch to surface water. A decrease in pumping by nearly 20 percent has caused water levels to stabilize in many areas with a consequent decrease in subsidence rates. The District has been challenged and upheld in the courts (Beckendorff v. Harris-Galveston Coastal Subsidence District, 1978).

1980 Arizona Groundwater Management Act

The 1980 Arizona Groundwater Management Act was based upon recommendations of the twenty-five member Ground Water Management Study Commission which included representatives from cities and towns, mining, agriculture, Indian communities, and electric utilities (Session Laws-Arizona, 1980). The act mentions subsidence only three times (§45-402, §45-569, §45-603), but because the act is designed to establish safe yield in ground-water basins, measures that will control ground-water-level declines will control subsidence.

The act provides for Active Management Areas (AMA's) and Irrigation Non-Expansion Areas (INA's) administered by regional offices of the newly formed Department of Water Resources. Four initial AMA's cover Tucson, Phoenix, Prescott, and Pinal County (§45-411A). These four areas contain more than 80 percent of the State's population and account for about 70 percent of the State's ground-water overdraft (J. W. Johnson, attorney, written commun., 1981). The State director may designate subsequent AMA's, after public hearings and with due process, if it is determined that, among other reasons, land subsidence or fissuring is endangering property or potential ground water storage capacity, (§44-412). The director may designate subsequent INA's if "There is insufficient ground water to provide a reasonably safe supply for irrigation of the cultivated lands in the area at the current rate of withdrawal," and "...establishment of an active management area...is not necessary," (§45-432). There are provisions for local designations of AMA's and INA's (§45-415, §44-433) and conversion from INA's to AMA's (§45-439).

The target date for safe yield in the Tucson, Phoenix, and Prescott AMA's is on or before January 1, 2025. The goal in the Pinal AMA is to develop non-irrigation uses while preserving existing agriculture as long as feasible. Five management periods: 1980-1990, 1990-2000, 2000-2010, 2010-2020, and 2020-2025 are to be used to implement increasingly stringent conservation measures. Conservation programs for non-irrigation uses of ground water include non-specific reductions in per capita use for municipalities. For industries, the program requires the use of "...the latest commercially available conservation technology consistent with reasonable economic return," (§45-564). Conservation for irrigated agriculture will be achieved by reductions in ground-water allotment, termed irrigation-water duty, based on the necessary quantity of water *needed to irrigate the crops historically grown in the farm unit, assuming* "...the maximum conservation consistent with prudent long-term farm management practices within areas of similar farming conditions, considering the time required to amortize conservation investments and financial costs," (§45-565). In the third management period "...the director may adjust the highest twenty-five percent of the water duties within the sub-basin to more clearly reflect the average of the middle fifty percent of the water duties within the sub-basin," (§45-566). Later management periods include programs for augmenting water supplies including incentives for artificial recharge (§45-565) and for purchase and retirement of grandfathered water rights (§45-567).

The act provides for levying and collection of a fee between $0.50 and $1.00 per acre-foot (1,233 m^3) of ground water pumped per year for administration and enforcement of the act, to as much as $2.00 per acre-foot per year for augmentation of the water supply of an AMA, and, no earlier than January 1, 2006, as much as $2.00 per acre-foot per year for purchase and retirement of grandfathered rights (§45-611).

Violators of the act are subject to cease and desist orders issued by the director, preliminary or permanent injunctions from the superior court, civil penalties, and criminal penalties. Civil penalties have a maximum of "Ten thousand dollars per day of violation directly related to illegal withdrawal, use, or transportation of ground water," and a maximum of $100 per day for other violations (§45-635). Class 6 felonies include falsifying or tampering with a measuring device and knowingly withdrawing

1,000 acre-feet or more of ground water in violation of the act. Illegal withdrawals of smaller quantities are misdemeanor offenses (§45-636).

The act has been challenged and upheld in the courts (Clifton N. Cherry v. Wesley E. Steiner, 1982). In Cherry v. Steiner, plaintiffs, who reside in the Prescott AMA, challenged in the United States District Court that the 1980 Arizona Groundwater Code violates due process and equal protection under the United States Constitution. The trial court granted summary judgement to the defendants. The United States Supreme Court refused without comment to hear the case. An important aspect of the court tests of the act is its non-severability: "If any portion of this act is finally adjudicated invalid, the entire act shall be null and void," (§45-401). This non-severability was included in the act to protect agricultural, municipal, and mining interests, all of which had made concessions to allow passage of the bill. In addition to court challenges of the statute, suits based on the statute are being brought (Cortaro Water Users' Association v. Wesley E. Steiner, City of Tucson, 1983). In this case, a partial summary judgement was granted for the plaintiffs who appealed from the Department of Water Resources' decision to issue drilling permits to the City of Tucson outside its service area.

Arizona Subsidence Monitoring Plan

The National Geodetic Survey, with advice from an interagency Land Subsidence Committee, has submitted a subsidence monitoring plan to the Governor of Arizona. The plan summarizes known subsidence in the State and recognizes hazards for canals, pipelines, wells, etc. caused by subsidence, differential subsidence, and earth fissures. The objectives of the plan are: "Documentation of the location and magnitude of existing subsidence and subsidence-induced earth fissures. Development of procedures for estimating future subsidence as a function of water level decline and defining probable areas of future fissure development." The plan proposes a central facility at a State agency for compilation and organization of leveling, compaction, gravity and other geophysical, and stratigraphic information. There would be coordinated analysis of existing data to produce initial estimates of future subsidence and earth-fissure development and identify additional observation requirements. Other provisions include: "An initial observation program designed to obtain a limited amount of additional leveling data, gravity observations, compaction measurements, and horizontal strain determinations. A cooperative effort between State and Federal agencies to evaluate new measurement technologies which offer the potential of being faster and more cost effective than current methods of subsidence monitoring." Also included are proposals for directions in research, some initial monitoring plans, and an advisory committee to oversee the formation of the central data facility and provide continuing guidance (Strange, 1983).

Summary

In Texas, California, and Arizona, mining of ground water has caused extensive subsidence. Damages in Texas are largely due to reactivation of faults and submergence. In Arizona, damages are mostly due to differential subsidence and fissuring. The legal approaches to abating subsidence vary in the different States. Recent legal remedies include the Harris-Galveston Coastal Subsidence District Act of 1975, the landmark Friendswood decision of 1978, which moderated the English rule of capture in Texas, conservation measures by California water districts, and minor provisions of the 1980 Arizona Ground Water Management Act. A governor's commission has produced a subsidence monitoring plan for Arizona.

Discussion

The effort to control subsidence and subsidence related damages is in its infancy. The effectiveness of measures to prevent or correct damages will become increasingly important as damages become more acute. Effective long-range planning is needed to arrive at corrective measures that equitably distribute risks and costs. In spite of agricultural opposition to internalization of costs, it appears that, in Texas, subsidence is beginning to be controlled. In Arizona, the State subsidence-monitoring plan deals only in a limited way with horizontal deformation and stress-strain modeling, and it does not address abatement or control of subsidence. However, the State agency that will probably administer the subsidence plan is the Department of Water Resources, which administers the Ground Water Management Act. A hypothetical case illustrates the potential damages. Annual water level declines in subsiding basins in Arizona range from less than 1 to more than 3 m yr^{-1}. If safe yield is not achieved until the year 2025 and the mean rate of water-level decline is halved in each of the management periods, then total expected ground-water-level decline might range from 20 to 60 m. Assuming a virgin specific compaction of 0.02 m of subsidence per meter of ground-water-level decline (Anderson and others, 1982; M. C. Carpenter, unpublished data, 1984), 0.4 to 1.1 m of subsidence could occur. Differential subsidence and earth fissures may form over shallow buried structures as a result of such subsidence.

Acknowledgments

We thank Stanley N. Davis and Thomas Maddock III, Department of Hydrology and Water Resources, University of Arizona; and Jacque Emel, Arizona Department of Water Resources, for thoughtful reviews. We also thank Carlena Smith for superlative manuscript preparation.

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SUBSIDENCE PHENOMENA DUE TO EARTHQUAKES: ITALIAN CASES

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Abstract

Subsidence phenomena are among the most important geodynamic variations, mainly due to their great extent. They may have a more or less slow development or, when triggered off by earthquakes, a very rapid evolution. The latter is the case of the modifications in the elevation of entire regions following very violent seismic events. Among the natural causes of these modifications, a basic role is undertaken by the seismo-tectonic component, whose primary effect are tectonic dislocations of the bedrock. In more superficial soils, another consequence not less important is given by minor dislocations caused by various sudden effects of the seismic action.

Among the noteworthy historical Italian cases that have been studied, the author cites the ground lowerings and the mass movements which occurred in the piedmont benches and in the Pleistocene terraces of Aspromonte (Calabria) during the ruinous earthquake of February 5-6, 1783, that caused the most marked geomorphological changes in the recent history of the region.

After briefly mentioning the effects of two other violent earthquakes occurred during this century (Messina - 1908 and Friuli - 1976), the author deals more in detail with the effects of the November 23, 1980 earthquake in southern Italy, that produced marked land level changes, as a result of the occurred tectonic fractures. In connection with the geostructural characters of the area most directly affected by the earthquake, the author shows the changes in elevation checked along two alignments of reference benchmarks located in the epicenter area. Excluding isolated cases of evident surface compaction, corresponding to some abnormal peaks, it is possible to speak of seismo-tectonic subsidence in the strict sense of the term.

As a consequence of a probable tilting of huge aquifers located in the Mesozoic carbonate masses, the big variations in flow rate, observed in some large springs of the stricken area, are greatly evidenced.

Introduction

A growth in the knowledge of structural geology, geophysics and geotechniques has led to giving seismic phenomena, and secondly volcanoes, a broader and determining role among the causes which in historical times contribute to ground elevation processes.

Even land subsidence has to be included among the geodynamic changes of paramount importance, mostly because of its great extent. Usually it proceeds in a more or less slow way, but when set in motion by earthquakes it vice versa develops very rapidly. This is the case of land elevation changes of entire regions as a consequence of very violent seismic events.

In fact, seismic activity occurs primarily through tectonic displacements of the bedrock; secondary displacements of nearly the same importance, add to the first in the upper soil. They are due to the multiple effects, sudden or not, such as the "décollement" of plastic formations formed by allochthonous nappes which might hide the deep-seated tectonics, the translation of klippe, the compaction of cover soil, the "liquefaction" phenomena of loose saturated soils, the collapsing of subterranean cavities, etc.

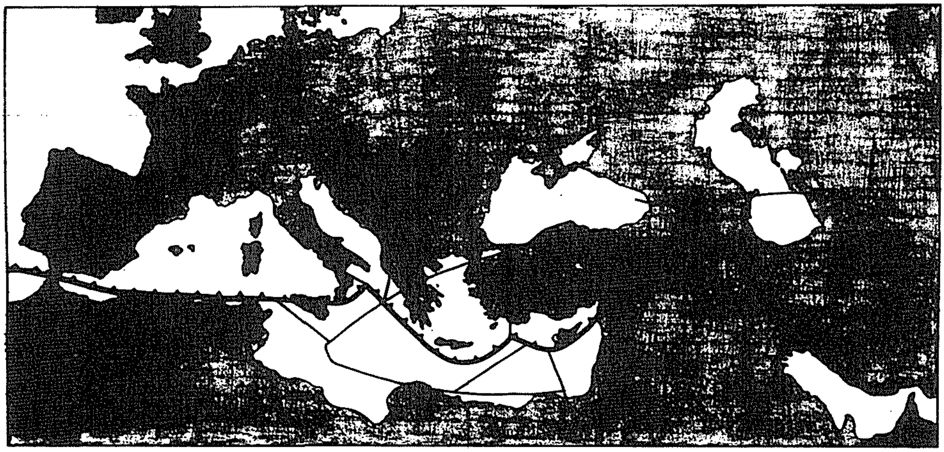


FIG. 1. Interaction between the continental plates of Africa and Euroasia.

Italian Case Histories

The correlation between seismic and subsidence events is very close, often with dramatic consequences in a place like Italy, which is crossed by the boundary between the Euroasiatic continental plate and the African one which underthrusts the other (Fig. 1). The adverse geological and the complex morphological conditions reveal themselves in frequent macroscopic situations of ground fragileness and instability.

It is known, in fact, that the great tectonic instability of the Italian peninsula arises from the interaction between the two lithospheric masses. Their convergence process created very distinct alignments of tectonic fractures, with the identification of microplates in related movement and the establishment of hypocenters of deep and surface earthquakes at the points of greatest friction.

The seismic activity is lesser in Italy with respect to other areas all over the world. In spite of this, the effects of seismic events with a high magnitude have always been catastropic owing to the consequent dramatic instability produced on the territory (sudden land lowerings, compaction, landslides, modifications in the hydrographic network, floodings, etc.).

The "Calabrie" earthquake,which occurred in February 1783, is one of the events in the distant past, whose effects have recently been studied. Lasting over 3 minutes, this earthquake is dolefully noted for having not only caused a horrifying decimation of human lives, but also the largest geomorphological upset that the region remembers. In particular, land sinkings were very notable, varying from a minimum of 3 m to a maximum of 6.6 m; this results from studies carried out immediately after the earthquake by various scientists and more recently taken up again.

Through documents from that period and the geomorphological disfigurations still recognizable after two centuries, it was possible to reconstruct the geological pattern for most of the epicentral zone, located on the northern side of Aspromonte (Fig. 2).

A great extent of the landslides, which at times occurred with displacements of a few hundreds of meters along the piedmont and the Pleistocenic terraces of Aspromonte, sweeping away entire villages, were determined. Very notable changes in the substratum elevation took place concurrently with the

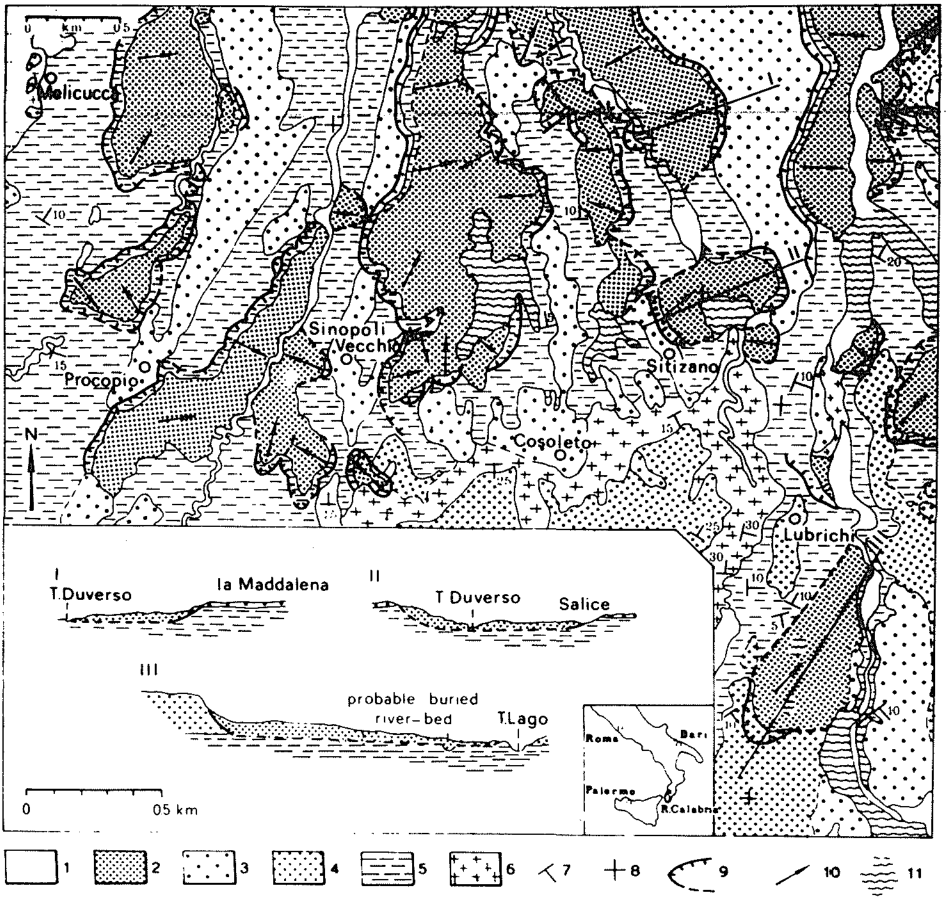


FIG. 2. 1783 Calabria earthquake: Detail of geological and landslide map. Most of the landslides occurred as the result of liquefaction produced by the earthquake. (1. Fluviatile deposits - Holocene; 2. Landslide debris; 3. Sands and conglomerates - Pleistocene; 4. Sands and sandstones - Pleistocene-Pliocene; 5. Clays and marls - Lower Pleistocene-Pliocene; 6. Metamorphics: gneiss and schists - Palaeozoic; 7. Attitude of strata; 8. Horizontal strata; 9. Main scarp and landslide limits; 10. Main direction of mass movements; 11. Lakes formed as a result of the landslide).

landslides, inducing diversion or damming of various water courses; as a consequence 215 lakes with lengths up to 2 km and depths down to 40 m were forme

In Italy since prehistoric times, striking earthquakes have been noted in the Reggio Calabria-Messina area. In particular for this century, the earthquake of a magnitude M=7, which occurred on 28 December 1908, is still very much remembered. The geodetic surveys carried out by the Istituto Geografico Militare Italiano (IGMI), before and after the occurrence, verify that the earthquake caused a subsidence of 70 cm in Messina and about 50 cm in Reggio Calabria (Caputo, 1978) (Fig. 3). A greater sinking is supposed to have oc-

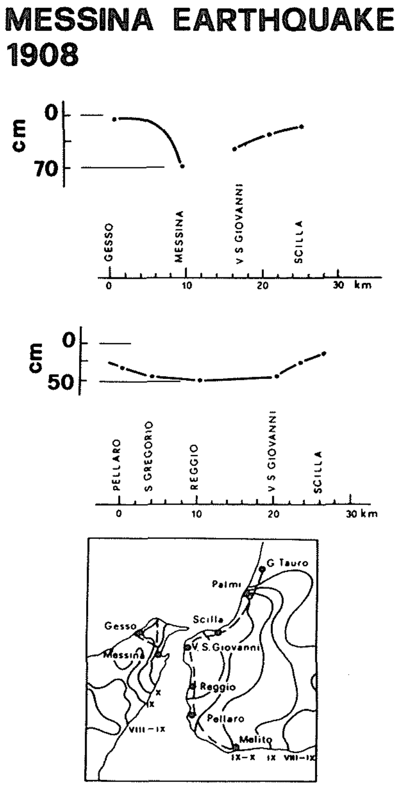


FIG. 3. Earthquake of Messina (1908): leveling lines made by IGMI before and after the earthquake and the recorded subsidence (after Caputo, 1979).

curred at the bottom of the strait. Such land elevation changes fit in rather well with the complex graben structured fold tectonics, recently defined in the area of the Strait of Messina by Selli (1979). On the basis of different seismic and volcanic parameters (Morelli, 1970, and Ritsema,1970), it is hypothesized that under the "calabro-eoliano" arc a lithospheric plate in subduction towards WNW is present.

Even the earthquake in Friuli on 6 May 1976 (M=6.4), besides the activation and re-activation of more than 200 landslides in faulty areas, caused serious vertical land displacements. A re-survey of the leveling networks in the area devastated by the earthquake was made by IGMI in 1977. With respect to the 1952 survey, it showed the existence of two very distinct zones located to the south and to the north of the epicenter, which decidedly were raised 18 cm and lowered 7 cm respectively; an overall variation equal to 25 cm then resulted (Talamo et al., 1978) (Fig. 4). Land sinkings even of 50-100 cm were surveyed in some restricted areas where a compaction of the normally consolidated postglacial sands added its effect to the lowering of the substratum (Siro, 1977).

Also the remarkable land elevation changes recorded after the earthquake occurred in southern Italy on 23 November 1980, must be regarded as widespread neotectonic events.

This 6.8 magnitude earthquake lasted 80 seconds and hit a large area of the Apennines between the Campania and Lucania regions. Their seismic activ-

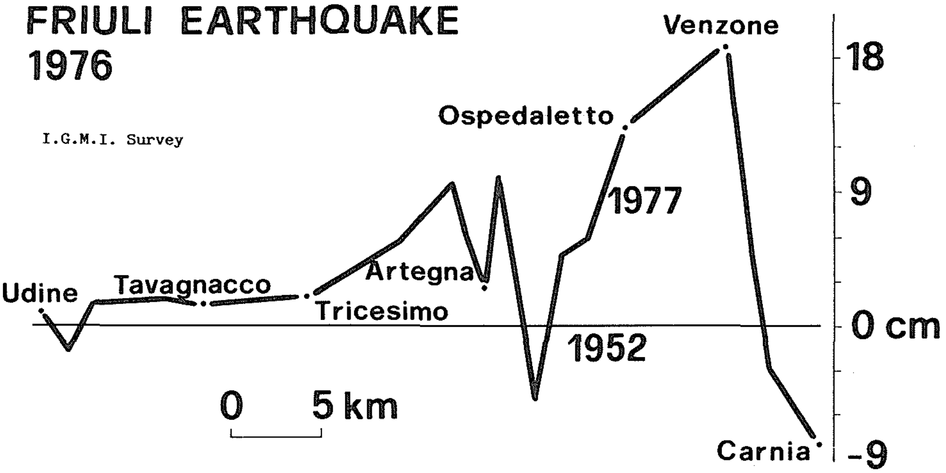


FIG. 4. Earthquake in Friuli (1976): ground elevation changes recorded by IGMI following the seismic event (after Talamo et al., 1978).

ity, connected with the persisting uplifting of the mountain chain, is one of the most intense and frequent of the Italian peninsula. The maximum intensity of the epicenter was X on the MSK scale, while the maximum acceleration recorded at the existing seismographic stations was along the E-W component and equal to 0.33 g.

The geostructural features of the western sector of the area most devastated by the earthquake show a predominance of the Mesozoic carbonate rock masses overthrusting the limy-marly series of the southern Apennines (Fig.5). Because of their tectonization, these calcareous and dolomitic rocks underwent large, widespread failures as a result of the seismic shaking.

The eastern sector is characterized by the presence of flysch formations; prevalent here is what is known as the Undifferentiated Sicilide Complex, with a predominance of "argille varicolori scagliose" (scaly multicolored clays). These clays have usually very poor geotechnical properties so the whole sector is generally unstable with broad and deep mass movements. Many extensive landslides were reactivated in the area; that of Caposele is shown for example in figure 6.

The Plio-Quaternary phases of tension tectonics produced the graben structures of the Sele and Ofanto valleys, which were extensively involved in recent seismic events. Some leveling lines crossing the F. Ofanto Valley and based on benchmarks placed in the stiff clays of the Pliocene Era were surveyed a year before and right after the earthquake; a subsidence of about 75 cm along a straight line distance of only 4.5 km was measured. Between the abutments of the Conza della Campania earth dam (under construction) there was a gradual lowering of the foundation base up to a maximum of 17 cm over a length of about 600 m.

In the lower portion of figure 5 the survey of some geodetic stations, carried out by the IGMI after the earthquake, is shown, along with the variations in height with respect to the 1959 survey as measured along two trigonometric benchmark lines falling within the epicentral area (Arca and Marchioni, 1983). A part from the isolated cases of evident surface compaction, corresponding to some abnormal peaks, one must speak about seismotec-

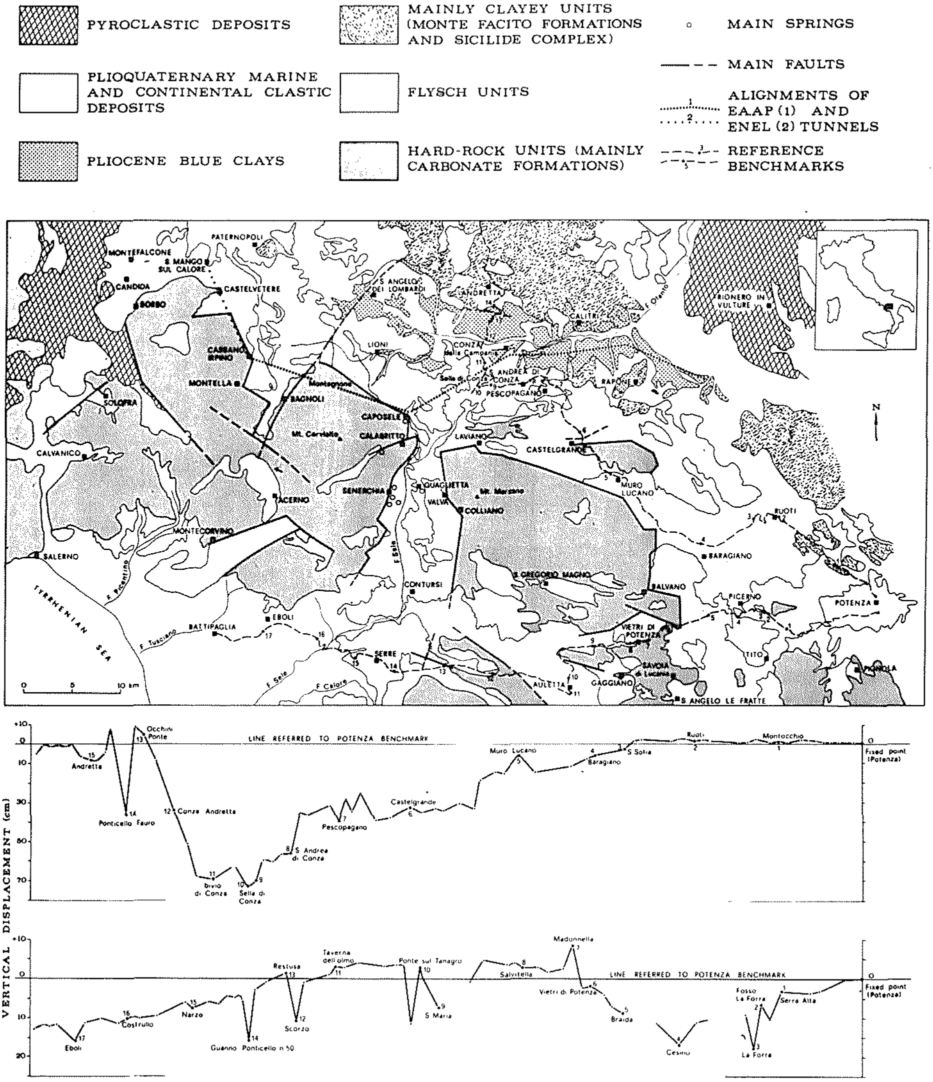


FIG. 5. 23 November 1980 earthquake in southern Italy. Geostructural sketch and vertical displacements checked in some geodetic bases by the Italian Military Geographic Institute (IGMI) as a result of the quake.

tonic land subsidence in the strict sense of the term, which reaches 80 cm. The altimetric profiles outline a number of steps, very likely corresponding to the fault intersections with geodetic stations.

In road embankments as well as in recent alluvial deposits, and especially in eluvial and colluvial sediments, a considerable geotechnical compaction --of up to 50 cm-- due to the compressibility of these soils under cyclical vertical loading added to the seismotectonic subsidence.

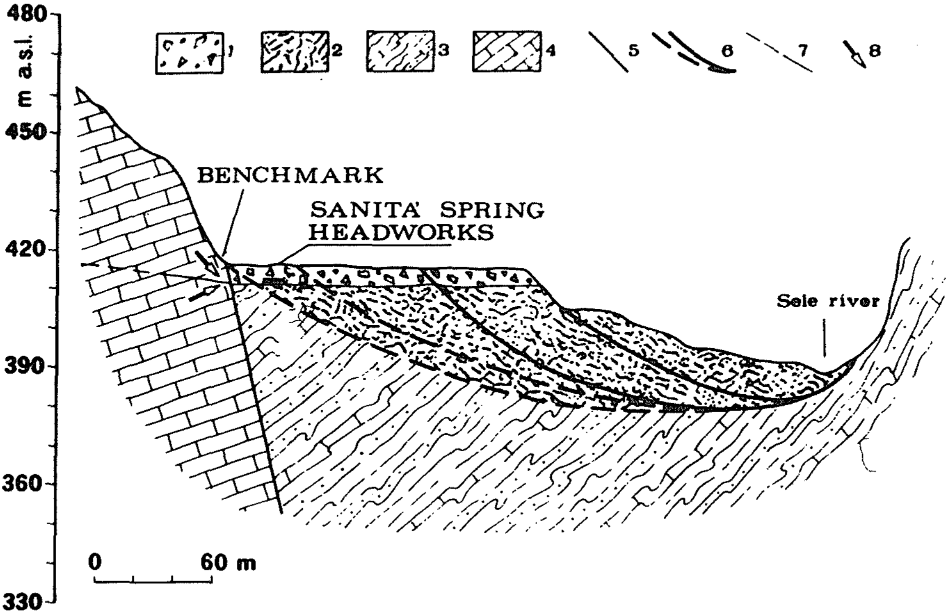


FIG. 6. Geological section of the Caposele area, located in the epicentral zone of the 23 November 1980 earthquake. 1) Sanità spring headworks yard fill material; 2) Landslide debris; 3) Limy-marly-sandy formation; 4) Mesozoic carbonate rocks ; 5) Fault; 6) Slip plane; 7) Piezometric level; 8) Direction of flow.

Even many engineering structures linearly running suffered great changes in height, while the strong deep-seated tectonic stresses caused serious damages to some tunnels of the hydroelectric plants and to the underground aqueduct. The geological profile of a portion of the "Pugliese" aqueduct, where the cover has a maximum thickness of 350 m, and some of the main damages it underwent during the earthquake are illustrated in Fig. 7. In some stretches the original slope has more than doubled.

More frequent and greater damages occurred where the "argille varicolori scalgiose" are present because of their very poor geotechnical properties and a very low strength. The greatest damage is the squeezing of the cover and related reduction of the free section. In the most severely damaged stretch, the invert was raised as much as 1.70 m. Failures given by sideways and longitudinal stresses are very evident where the scaly multicolored clays meet the Pliocene blue clays. These latter experienced only isolated breaks with throwing where there are faults.

The Apulian aqueduct,so seriously damaged by the 1980 earthquake, is supplied by large overflowing springs located at the faulted passage from the carbonate water-bearing strata to the deposits of flysch, multicolored clays and valley detritus. These springs underwent great changes in flow after the earthquake.

As apparent in the 1980-82 flow diagrams (Fig. 8), the flow rate of the Sanità Spring at Caposele, rose, within 40 days after the main shock from 4200 l/s to 7300 l/s with a maximum daily flowing rate of 400 liters/sec. The causes of such an exceptional behavior, not attributable to meteoric e-

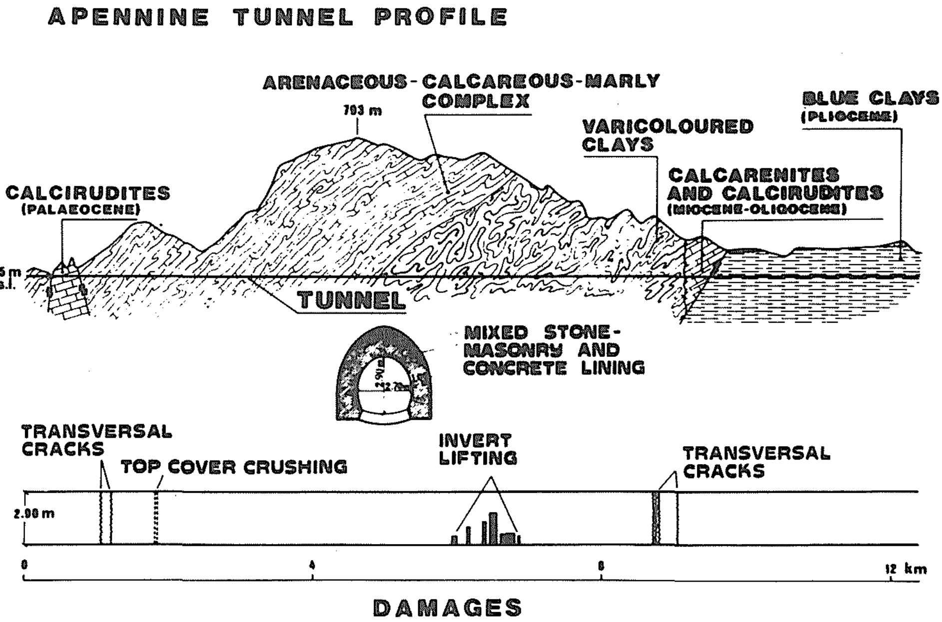


FIG. 7. Apennine (Pavoncelli) Tunnel of the Apulian aqueduct. Geological profile and different kinds of collapse features caused there by the 23 November 1980 earthquake. The lining was fit for supporting pressures exerted by the mountain up to 10^5 kg/m^2.

vents, are certainly in the lowering of the sill of the spring for the whole aquifer. In particular the barrier of the carbonate aquifer at the spring has lowered about 80 cm; this could indicate a probable tilting of the aquifer towards the spring.

In another group of springs, namely that of Cassano Irpino, the rapid flow increase started about two weeks before the seismic occurrence as a sign of the earthquake.

In some Italian areas the high seismic risk is joined with a great risk of volcanic activity which is in turn responsible for some land elevation changes: these phenomena are active today in the Phlegraean Fields ("Fiery Fields") in Campania.

The Phlegraean Fields are involved in a tension tectonic which in recent geological and historical times produced several surface eruptions. The remains of the old Phlegraean lie buried under lava and pyroclastic materials from numerous volcanoes, the last of which, the Monte Nuovo one, emerged in 1538 from an explosion reaching a height of 140 meters within a few days.

The Phlegraean Fields are indeed a large scale example of bradyseism presently under stress. It is due to the evolution of magma intrusion at shallow depths. Pressure increases with decreasing temperature and magma appears differentiated, light and rich in gas. Such cases are often affected by phreato-magmatic events, due to water infiltration from the aquifer, or from

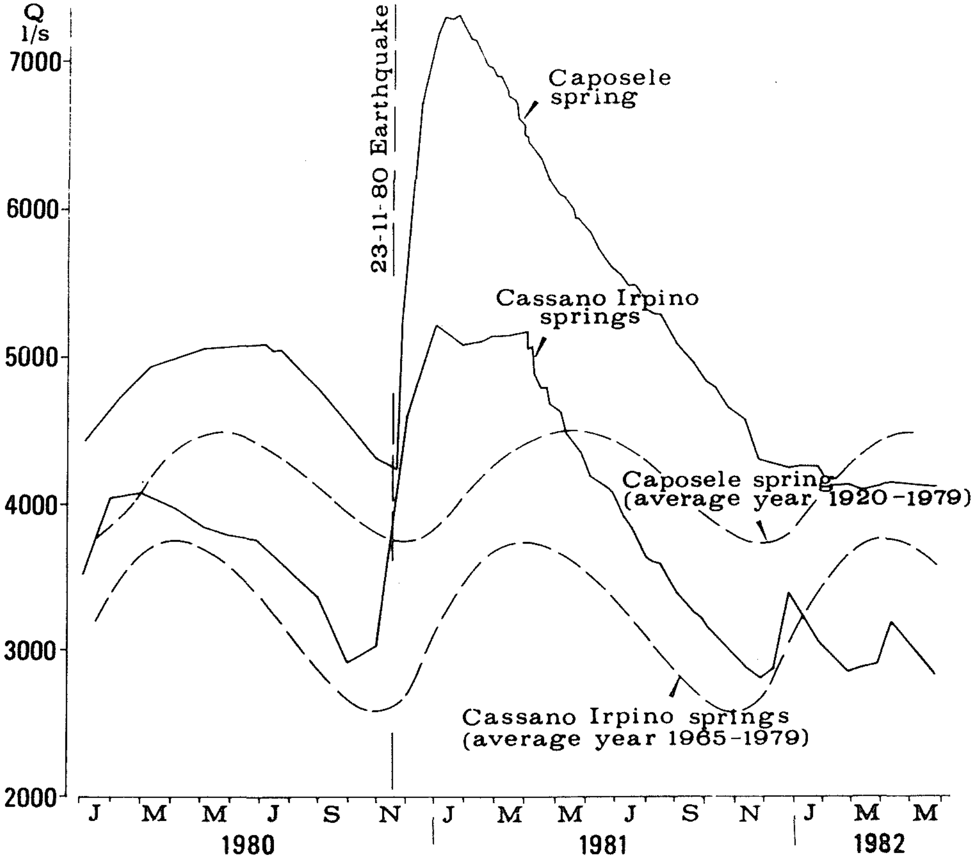


FIG. 8. Discharge hydrographs of the Sanità di Caposele spring and the Cassano Irpino group of springs before and after the 23 November 1980 earthquake. Comparison of these hydrographs with those for the average year reveals the big increase in discharge as a result of the quake.

the sea, into the cracks surrounding the body. Evaporation processes, sudden pressure increases, enlargement of cracks and friction phenomena, the penetration of magma in these cracks, earthquakes, swelling of the ground and sometimes rythmic violent eruptive explosions followed by long periods of subsidence, are the result.

In the Phlegraean Fields the top of the magma body is nearly 3 kilometers below ground level; it contains hydrothermal pipes, among them the active crater of Solfatara is very notable.

For 2000 years the columns of the Temple of Serapis represent the most valuable metric index to assess bradyseism at Pozzuoli. The highest sea level reached during the downward phases of bradyseism is about 6 m above the columns base.

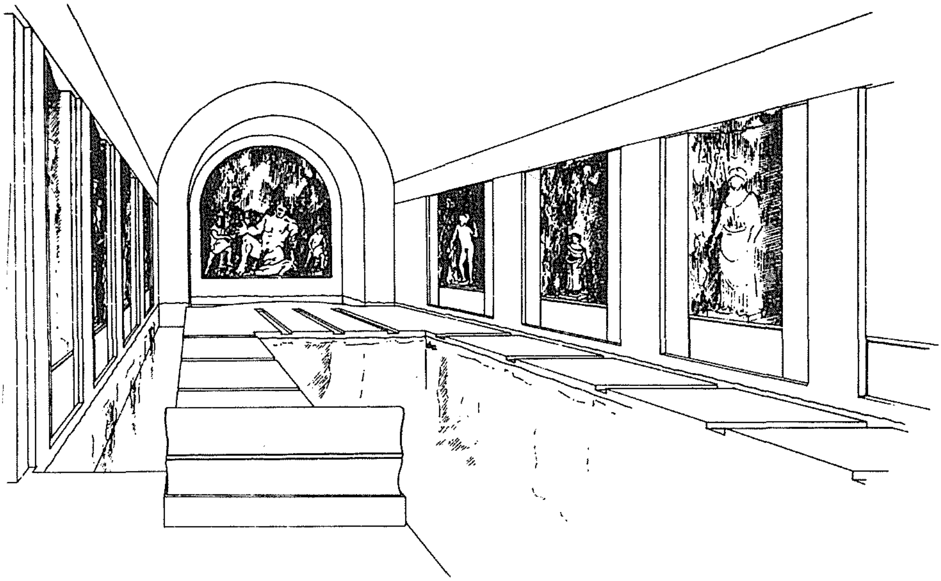


FIG. 9. Axonometric drawing of Emperor Claudius' Palace at Bais (about A.D. 45) (after Andreae, 1983).

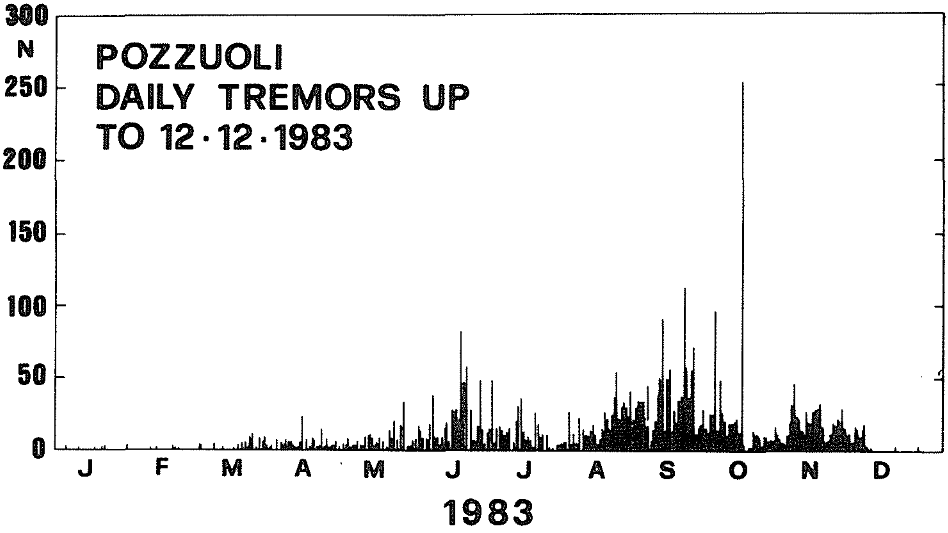


FIG. 10. Pozzuoli. Daily distribution of seismic events up to 12 December 1983.

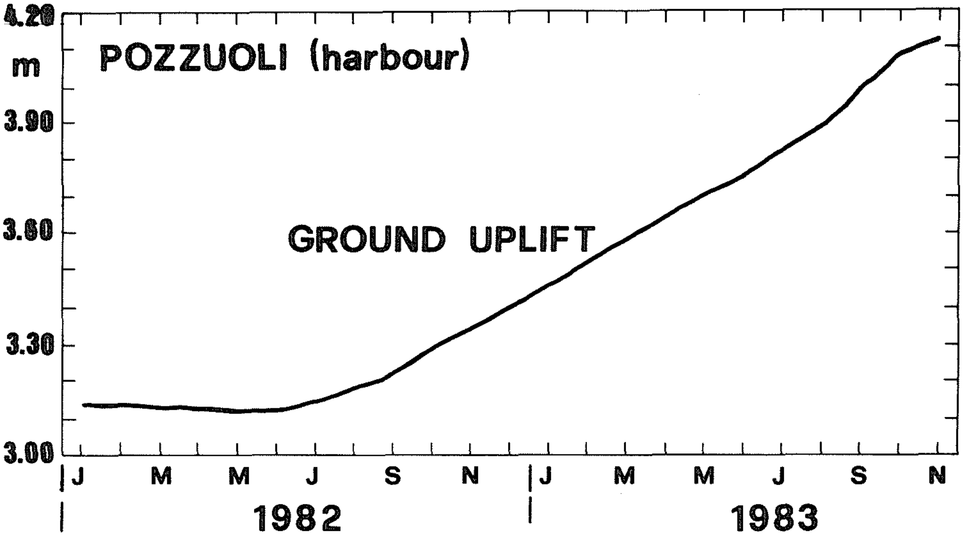
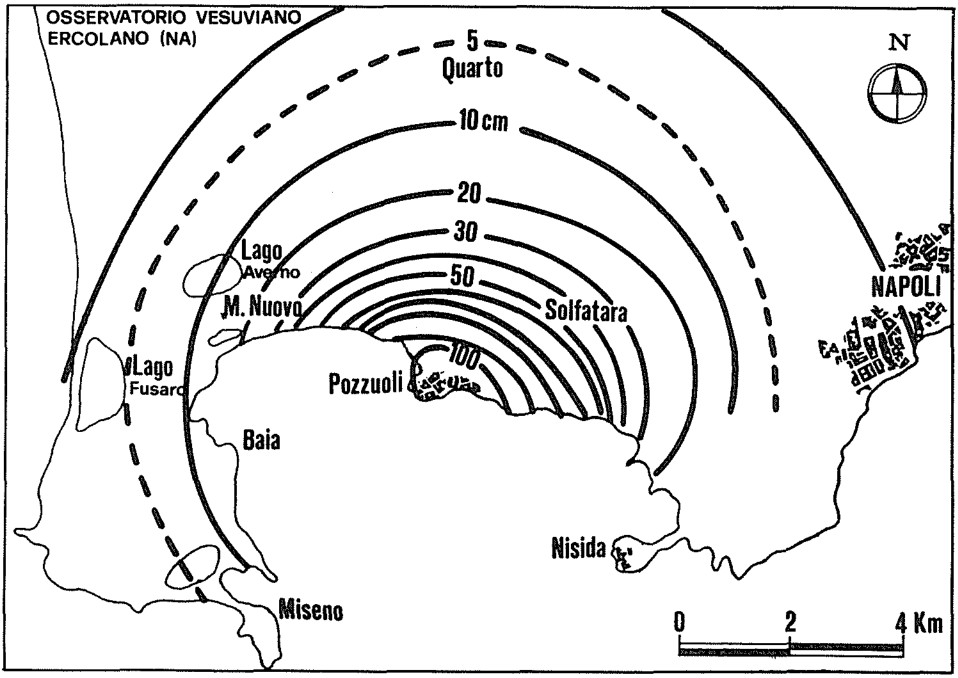


FIG. 11. Pozzuoli. Raising of the ground in the harbor area.



ISOKINETIC LINES : January 1982 - December 1983

FIG. 12. Pozzuoli. Isokinetic lines showing the areal distribution of the uplifting.

An archaeological testimony of this phenomenon is provided by the recent discovery of the ruins of the Roman imperial palace at Baia, a neighboring city of Pozzuoli, on the seafloor about 7 m deep. It is ascertained that an entire section of the Roman city of Baia, located along the coast, sunk under the sea because of a 14 m of subsidence. In particular, the remains were found of a large aspe-like room of the Palace of Emperor Claudius, dating back to A.D. 45 (Fig. 9). Various statues were brought to light among which those of Ulysses and one of his companions; they originally were part of the marble statues depicitng the legend of Polifemo. The dating of the discoveries establishes that land subsidence took place around A.D. 300, probably accelerated by earthquakes. In the sixth century the area reemerged and then sunk permanently to its actual depth of 7 meters (Andreae, 1983).

Nowadays a very strong increase in the Phlegraean phenomenon, is occurring. After an extremely long period characterized by a low unrelevant subsidence, an abrupt reversal of trend has been observed from 1969. The raising of the ground reached in that year a rate of 5 mm/day, involving the zone which extends from the offshoots west of Naples to Cape Miseno, all around the town of Pozzuoli. With respect to the 1953 ground elevation, land surface was raised to a maximum of 1.50 m. This uplift caused collapsing of buildings, the abandonment of city districts of Pozzuoli and extensive damages to the port structures. A notable seismic activity occurred concurrently in the area.

In fact from November 1982 to October 1983, over 2000 earthquake shocks of a variable intensity up to VII on the Mercalli scale were recorded (Fig. 10). Their hypocenters were at a depth between 2 and 4 km. The greatest shock recorded on 4 October 1983, had a magnitude M=4.0. The seismic events show themselves in several clusters indicative of the heterogeneity of the Phlegraean subsoil. The most numerous cluster refers to 13 October 1983 with 250 tremors which occurred within about 5 hours. With respect to 1970 a further 110 cm ground raising occurred between January 1982 and December 1983 at an average speed of 2 mm/day, with peaks of 4-5 mm/day (Fig. 11). The isoseismal lines for this period are reported in figure 12. From 1970 to 1983 the land was raised a total of 2.60 m. Owing to these seismic events and the risk following, the built-up areas of Pozzuoli have been today evacuated.

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LAND SUBSIDENCE AND EARTH FISSURES CAUSED BY GROUNDWATER DEPLETION IN SOUTHERN ARIZONA, U.S.A.

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Abstract

Land subsidence and earth fissures caused by ground-water depletion have affected more than 7,770 km^2 in southern Arizona. The subsidence has occurred as a consequence of aquifer compaction that resulted from declines in ground-water levels. Differential land subsidence has in turn produced extensive areas of earth fissures.

Land subsidence and earth fissures have damaged a variety of engineering structures including buildings, streets, highways, railroads, earthern dams, water wells, water-distribution systems, and sewage-disposal facilities. Other effects include an increased potential for pollution, increased flood hazards, and accelerated erosion.

Introduction

Land subsidence and earth fissures caused by ground-water depletion present geologic hazards in many parts of southern Arizona. Land subsidence has affected more than 7,770 square kilometers including parts of Arizona's two largest metropolitan areas—Tucson and Phoenix (Strange, 1983). Differential land subsidence and earth fissures have damaged a variety of engineering structures including buildings, streets, roads, highways, railroads, earthen dams, water wells, water-distribution systems, and sewage-disposal facilities. Fissures have caused rerouting of a part of the Central Arizona Project aqueduct system and adversely affected urban and agricultural land use.

This paper describes the general geohydrologic conditions in southern Arizona including historical ground-water withdrawals, water-level declines in wells, earth-fissure distributions, and amounts of measured land subsidence. Environmental effects of land subsidence and earth fissures and the relation between water-level changes in wells and resultant aquifer compaction and land subsidence are also discussed.

The study area is characterized by broad alluvium-filled valleys, bounded by rugged mountains that consist mainly of igneous, metamorphic, volcanic, and consolidated sedimentary rocks. The valley floors are extensively irrigated and are underlain by permeable unconsolidated to moderately consolidated alluvium that stores large volumes of ground water.

Annual precipitation ranges from less than 70 millimeters in the southwest corner of Arizona to slightly more than 500 millimeters in the central mountains. Potential evapotranspiration is more than 1,500 millimeters per year in most of the area (Environmental Data and Information Service, 1981). Although precipitation is sufficient to support some range grass and desert vegetation, irrigation is necessary to grow crops in the study area. About 555,000 hectares of crops—valued at $943 million—were grown in Arizona in 1981 (Valley National Bank of Arizona, 1982). About 90 percent of the crops were grown in southern Arizona.

Large-scale pumping of ground water began about 1900 and increased in the late 1940's. By 1981, about 233,000 cubic hectometers of ground water had been withdrawn from the alluvial aquifers (U.S. Geological Survey, 1982). Ground water is used mainly to irrigate crops, although municipal and industrial water use is increasing. About two-thirds of the ground-water withdrawal occurred in the Salt River Valley (SRV) and the lower Santa Cruz (LSC) ground-water areas (Fig. 1).

Ground-water withdrawals from the alluvial aquifer in the study area have greatly exceeded recharge, and most of the water pumped was removed from storage. Annual pumpage rates may exceed natural-recharge rates by more than 500 times in some areas (Arizona Water Commission, 1975).

Water-Level Change and Aquifer Compaction

Ground-water depletion has caused water levels in wells to decline throughout most of the study area (Fig. 2). Locally, water levels have declined as much as 140 meters (Laney and others, 1978). Water-level declines have caused compaction of silt and clay layers within the aquifer and has resulted in significant amounts of land subsidence. Differential land subsidence has produced extensive areas of earth fissures (Fig. 2).

The relation between water-level change, aquifer compaction, and land subsidence in Arizona was first documented at the vertical-extensometer installation near the town of Eloy in the lower Santa Cruz basin (Schumann and Poland, 1970). Compaction and expansion of the aquifer materials between the land surface and a depth of 253 meters correspond to seasonal trends in water-level fluctuations (Fig. 3). Aquifer compaction occurs during summer periods of water-level decline, and lesser amounts of expansion or rebound occur during winter periods of water-level recovery. Measured land subsidence corresponds to net annual water-level declines (Fig. 3). From March 1965 to December 1979, the land surface subsided 1.09 meters, and the compaction of sediments between the land surface and the bottom of the well near Eloy was 0.66 meter. Thus, it can be deduced that 0.43 meter of aquifer compaction occurred in sediments below the 253-meter depth of the well.

Vertical-extensometer installations are being used to measure and monitor aquifer compaction and water-level changes at 14 sites in southern Arizona. Six of the installations are along the route of the Central Arizona Project aqueduct system (Schumann and others, 1983).

Land Subsidence

An extensive review of leveling information by Strange (1983) indicates that measurable land subsidence has been detected in nine ground-water areas. The maximum amounts of measured land subsidence and water-level decline in each of these areas are listed in table 1.

Land subsidence was first detected in southern Arizona in 1948 through releveling of bench marks near Eloy in the lower Santa Cruz basin (Robinson and Peterson, 1962). Subsequent investigations indicated that land subsidence was continuing and that the area of greatest subsidence corresponded to the area of greatest water-level decline (Schumann and Poland, 1970). By 1977, about 1,100 square kilometers in the Stanfield area and 1,750 square kilometers in the Eloy area were affected by land subsidence (Fig. 2).

The maximum amount of subsidence was 3.8 meters near Eloy. As much as 3.6 meters of subsidence was measured near Stanfield (Laney and

EXPLANATION

Areas and abbreviations	Pumpage per area, in cubic hectometers × 10^3
ARA = Aravaipa Valley	0.0111
AVR = Avra Valley	4.76
BUT = Butler Valley	0.0419
CHI = Colorado River, Hoover Dam to Imperial Dam	0.758
DOU = Douglas basin	3.27
GIL = Gila Bend basin	8.05
GRD = Gila River drainage from Painted Rock Dam to Texas Hill	4.09
GTD = Gila River from Texas Hill to Dome	6.26
HAR = Harquahala Plains	4.17
HAS = Hassayampa basin	0.0148
LHA = Lower Hassayampa	0.917
LSP = Lower San Pedro basin	1.03
LSC = Lower Santa Cruz basin	53.3
MMU = McMullen Valley	3.03
NAZ = Northern Arizona	6.65
N-C = New River-Cave Creek	0.00863
RAN = Ranegras Plain	0.555
SAF = Safford basin	5.34
SRV = Salt River Valley	98.6
SSI = San Simon basin	3.22
SSW = San Simon Wash	0.0197
USP = Upper San Pedro basin	0.890
USC = Upper Santa Cruz basin	10.7
WAT = Waterman Wash	1.96
WMD = Western Mexican drainage	--------
WIL = Willcox basin	8.09
YUM = Yuma	7.34
Total pumpage	233

10 0 CUBIC HECTOMETERS X 10^3

GROUND-WATER AREA

FIG. 1 Ground-water areas and withdrawals in southern Arizona.

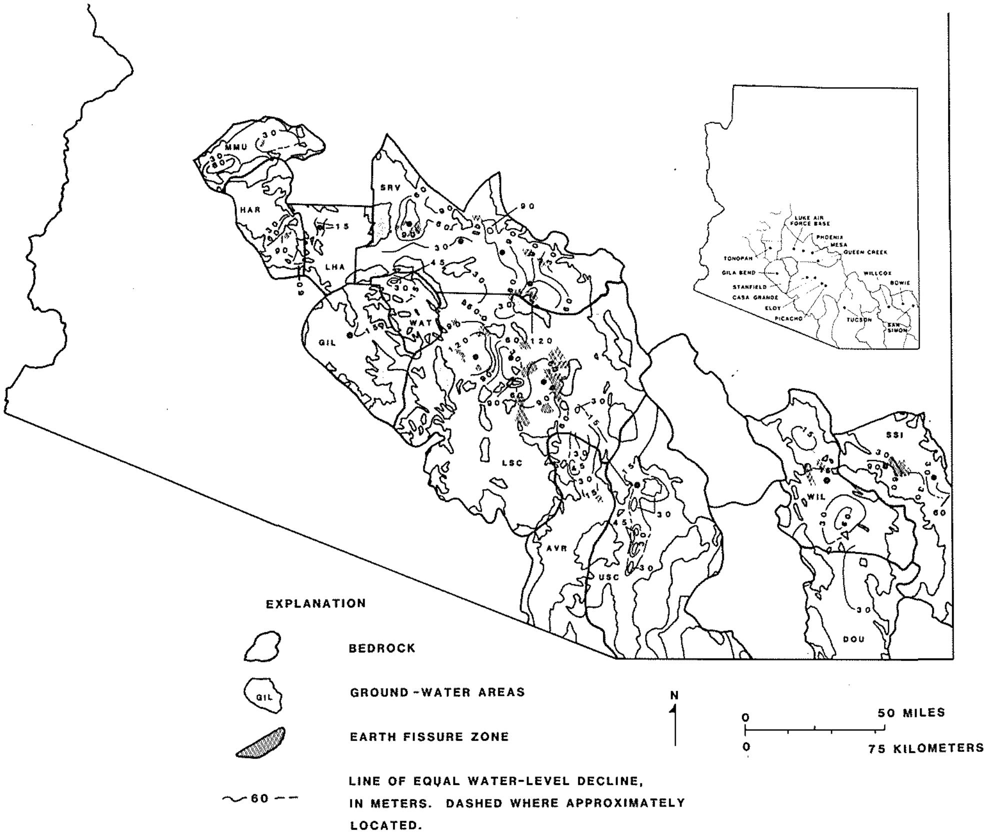

FIG. 2 Water-level declines and earth-fissure zones in selected ground-water areas of southern Arizona.

others, 1978). Leveling data collected in 1980 and vertical-extensometer data indicate that land subsidence and aquifer compaction are continuing in the lower Santa Cruz basin.

In the western part of Salt River Valley, an area of 400 square kilometers near Luke Air Force Base west of Phoenix and an area of 600 square kilometers near Queen Creek were reported to have subsided more than 1 meter by 1977 (Poland, 1981). Water-level declines prior to 1977 in both areas exceeded 100 meters (Fig. 2). Between 1948 and 1981, about 1.6 meters of subsidence was measured east of Mesa where water-level declines exceeded 100 meters (Strange, 1983).

In the eastern part of Salt River Valley, about 65 square kilometers subsided in the Paradise Valley area northeast of Phoenix where subsidence rates as great as 107 millimeters per year were measured (Harmon, 1982). As much as 1.5 meters of subsidence were reported in northeast Phoenix between 1962 and 1982 (Pewe and Larson, 1982).

Land subsidence is of great concern in the lower Santa Cruz basin where ground water is the only source of water for municipal, industrial, and agricultural use. About 0.13 meter of subsidence was measured between

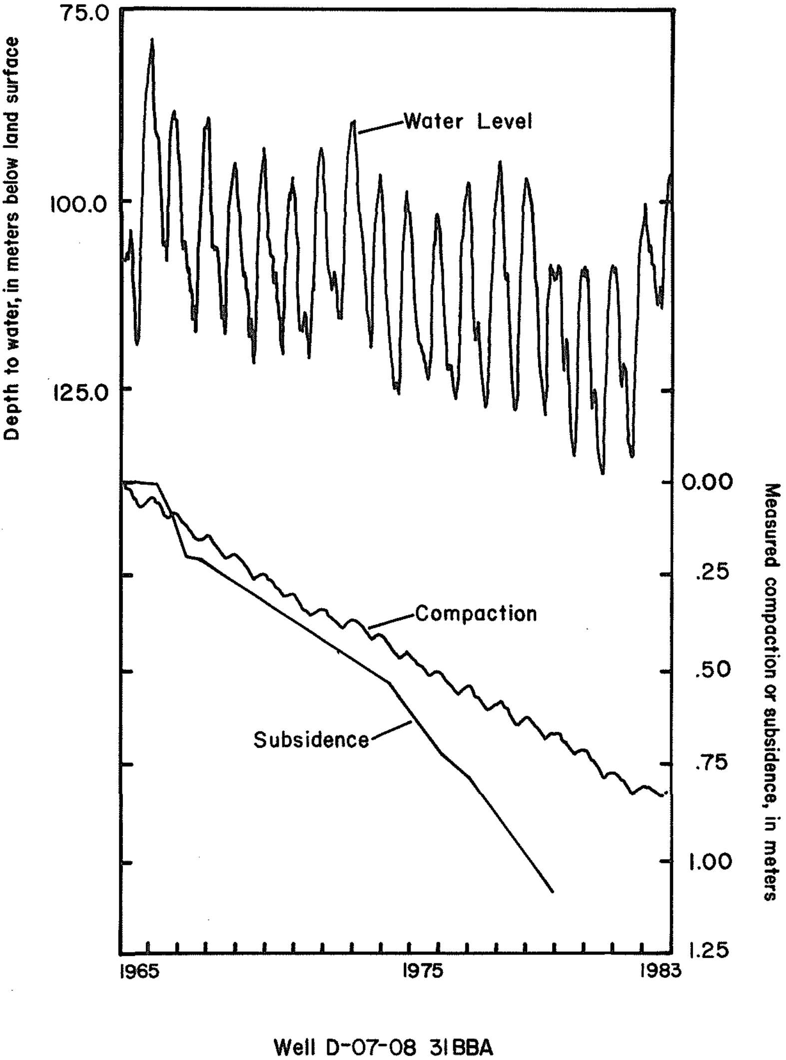

FIG. 3 Water-level changes, aquifer compaction, and land subsidence.

1952 and 1980, and water levels in wells declined more than 33 meters (Strange, 1983). Data from six vertical-extensometer installations indicate that water-level declines and aquifer compaction are continuing in the Tucson basin (Anderson and others, 1982).

Leveling data along the northwestern part of Avra Valley (Fig. 2) indicate that 0.32 meters of subsidence occurred between 1948 and 1980. The maximum water-level decline in wells in Avra Valley was more than 60 meters (Strange, 1983). A comparison of leveling data to bench marks measured in 1927 and 1981 indicates that 0.18 meter of subsidence may have occurred in the Harquahala Plains. The maximum water-level decline in wells in Harquahala Plains was more than 90 meters (Strange, 1983). Water

Table 1 Maximum amounts of land subsidence and water-level decline.

Ground-water area (See Fig. 1 for location)	Maximum water-level decline (meters)	Maximum land subsidence (meters)	Earth fissures
Lower Santa Cruz basin (LSC)	140	3.8	Mapped
Salt River Valley (SRV)	120	1.6	Mapped
Upper Santa Cruz (USC)	45	.13	Not reported
Avra Valley (AVR)	60	.32	Mapped
Harquahala Plains (HAR)	90	.18	Mapped
Willcox basin (WIL)	60	1.6	Mapped
San Simon basin (SSI)	90	1.78	Mapped
Lower Hassayampa-Tonopah basin (LHA)	15	.08	Not reported
Gila Bend area (GIL)	15	.08	Not reported
McMullen Valley (MMV)	60	----	Mapped

levels declined 15 meters in the Willcox basin, and about 1.6 meters of subsidence occurred northwest of Willcox (Holzer, 1980). Southeast of Willcox, water levels declined 60 meters and small amounts of subsidence occurred (Fig. 2).

A comparison of leveling data for 1952 and 1974 indicated that 1.42 meters of subsidence occurred near Bowie in the San Simon basin (Holzer, 1980). A comparison of leveling data for 1952 and 1980 indicated that 1.78 meters of subsidence occurred near Bowie and 0.39 meters of subsidence occurred near the town of San Simon (Strange, 1983). Water-level declines in wells exceed 90 meters near Bowie and 60 meters near San Simon.

Less than 0.08 meter of land subsidence was reported near Tonopah in the lower Hassayampa area and in the Gila Bend basin (Strange, 1983). Although 60 meters of water-level decline was measured in McMullen Valley, releveling data are not available to document land subsidence (Strange, 1983). Large water-level declines and the presence of earth fissures, however, indicate that land subsidence is probable in McMullen Valley.

Earth Fissures

Earth fissures occur in the alluvium mainly near the edges of seven major ground-water areas that are experiencing ground-water depletion. The fissures tend to connect and form linear fissure systems; in the lower Santa Cruz basin, one of the fissure systems is 15 kilometers long (Fig. 2).

Earth fissures first appear as narrow cracks that generally are less than 25 millimeters wide or as an alignment of shallow holes or sink-like depressions that are typically less than 75 millimeters in diameter. The new fissures have nearly vertical sides, generally exhibit no evidence of lateral or vertical movement, and range from a few meters to more than 1 kilometer in length (fig. 4). The simple horizontal separation of the landblocks indicate the fissures are probably tensional breaks (Schumann and Poland, 1970). Vertical offset has been observed along a few fissures after the initial break occurred.

Where an atypical earth fissure crosses Interstate Highway 10 about 5 kilometers east of Picacho in the lower Santa Cruz basin, about 1 meter of vertical offset occurred. The landblock is down to the west toward the center of subsidence. About 0.3 meter of offset has occurred in a block about 20 meters wide between a pair of subparallel fissures in the southern part of the Salt River Valley near Queen Creek (Fig. 2).

Fissures generally occur on the periphery of the basins, transect natural drainage patterns, and capture large volumes of surface runoff. Water flowing into and along the fissures produced gullies as much as 5 meters deep and more than 15 meters wide. The fissures enlarge by slumping, by erosion along the upstream sides, and by rapid piping along the strike of the fissures. Excavations across some of the fissures to depths of about 5 meters indicate that the fissure and associated pipes generally are filled with silt and sand that washed into the fissures.

Surface-geophysical investigations and deep test drilling indicate that some of the fissures formed over buried ridges or other irregularities in the consolidated bedrock surface beneath the unconsolidated to moderately consolidated alluvium (Jachens and Holzer, 1979; Schumann and Tosline, 1983). Differential land subsidence and lateral separation of the land blocks were measured at earth-fissure sites in northeastern Salt River Valley and lower Santa Cruz basin (Holzer and Pampeyan, 1981). The greatest concentration of earth fissures is in the lower Santa Cruz basin where the greatest amounts of water-level decline and land subsidence were measured. Earth fissures also were mapped or reported in six other areas including Harquahala Plains; McMullen, Salt River, and Avra Valleys; and Willcox and San Simon basins (Fig. 2). Significant amounts of ground-water depletion occurred in these areas, and water levels in wells continue to decline.

Environmental Effects of Land Subsidence and Earth Fissures

Environmental effects of land subsidence and earth fissures include actual and potential damage to engineering structures, accelerated erosion along earth fissures and drainages, increased flood hazards, and an increased potential for ground-water pollution. Damaged engineering structures include streets, highways, and railroads. Vertical offsets at the fissure site east of Picacho required regrading of the highway roadbed and the addition of ballast along the railroad (Winikka and Wold, 1977; Jachens and Holzer, 1979). Land subsidence and earth-fissure damage are important factors to be evaluated during the design of major engineering structures in areas of measured land subsidence.

Water and sewer systems have been damaged by land subsidence and (or) earth fissures. Water-well casings have collapsed as a result of compression, and expensive repairs and (or) replacement of irrigation and municipal wells has been necessary. Irrigation canals and buried irrigation pipelines have been broken. Large earthen dams have been damaged in or near subsiding areas. Releveling of irrigated fields in the lower Santa Cruz basin and in the Salt River Valley has been necessitated by land subsidence. Land subsidence has decreased the gradient of sewer lines in northeast Phoenix, which may require the installation of expensive pumping systems and chemical treatments (Harmon, 1982).

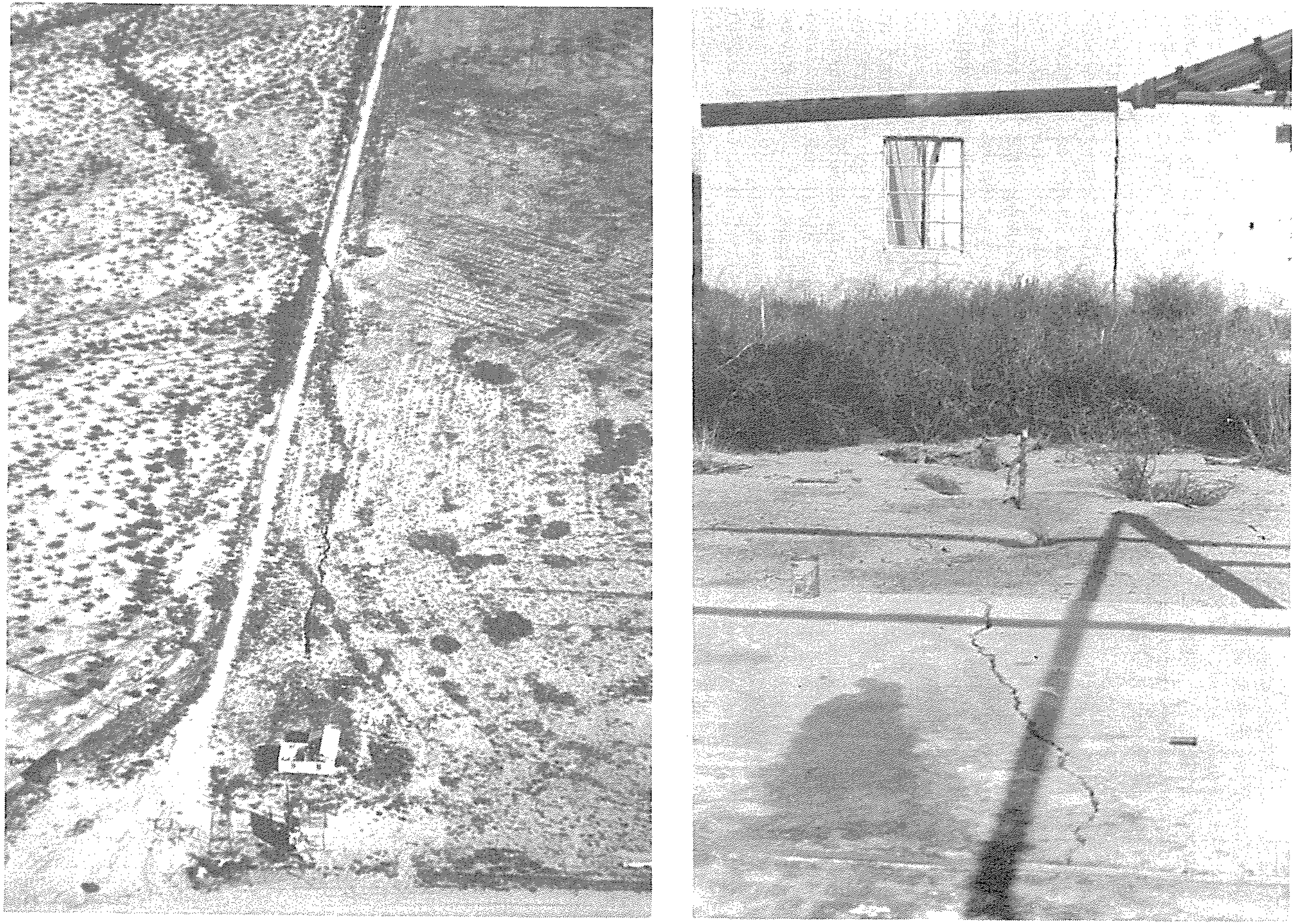

FIG. 4 Earth fissures in central Arizona.

Other problems with engineering structures include well casings that protrude above the land surface and cause damage to concrete pump bases. Homes in rural areas were damaged by earth fissures and subsequently were abandoned (Fig. 4). During flooding in the lower Santa Cruz basin in October 1983, a large earth fissure about 5 kilometers east of Picacho reopened and left about 15 meters of pipelines unsupported.

Accelerated erosion along earth fissures can form gullies as much as 5 meters deep and 15 meters wide. Open fissures of this size result in the loss of valuable irrigated land and present serious hazards to people, wildlife, and livestock. Accelerated erosion also occurs in natural drainages along the periphery of subsiding basins where the gradient between the basin floor and the surrounding mountains is increased. Accelerated erosion removes topsoil and deepens ephemeral stream channels.

In contrast, subsidence decreases the gradient along the lower reaches of major rivers and streams that traverse the subsiding basins. The reduced gradient of stream channels leaving the subsiding basin can create a backwater effect and cause the floodwater to spread out within the basin.

Agricultural and urban lands in the lower Santa Cruz basin were flooded to record depths in early October 1983. More than $50 million in damage was reported in these areas; some of these areas had subsided from 1 to nearly 4 meters (Federal Emergency Management Agency, 1983).

Pashley (1961) reported that a small fissure in the lower Santa Cruz basin diverted the entire flow of an irrigation ditch and captured about 1,230 cubic meters of water and several cubic meters of sediment over an 18-hour period before the fissure began to overflow. In the eastern Salt River Valley, a fissure system less than 500 meters long accepted roughly 7,700 cubic meters of water and about 76 cubic meters of sediment over a 5-day test period.

Another environmental consideration is the increased potential for ground-water pollution. Earth-fissure depths as great as 20 meters have been measured (Schumann and Poland, 1970). The earth fissures, however, may extend to greater depths and perhaps even to the water table. If the fissures extend to the water table, they could provide a direct path for surface runoff to the water table eliminating the natural filtration process through sediments in the unsaturated zone. Pesticides, herbicides, chemical fertilizers, and animal wastes may be contained in surface runoff from agricultural lands. Pipelines carrying sewage or toxic materials, or accidents that involve spillage of toxic materials from trucks or trains, also may be potential sources of ground-water pollution at or near earth fissures. Trash and (or) other material that have been dumped or washed into earth fissures at many sites may present public health hazards.

Summary and Conclusions

Large-scale withdrawals of ground water to meet the needs of extensive irrigated cropland and rapid population growth in southern Arizona have resulted in a rapid depletion of the alluvial aquifer. Water-level declines of more than 140 meters have caused extensive aquifer compaction, which has resulted in as much as 3.8 meters of measured land subsidence.

Measurable land subsidence has been reported in nine ground-water areas in southern Arizona, all of which have experienced ground-water depletion. Land subsidence is suspected in other areas of ground-water depletion.

Differential land subsidence has produced extensive areas of earth fissures in the alluvium in seven of the ground-water areas. Many of these fissures are believed to be tensional breaks formed over irregularities in consolidated sediments or bedrock beneath the unconsolidated to moderately consolidated alluvium. Differential land subsidence and earth fissures have damaged engineering structures and have adversely affected land use.

Accelerated erosion along earth fissures and natural drainageways presents serious problems for urban and rural land use. Earth fissures may increase the potential for ground-water pollution by providing a direct path for surface runoff to reach the water table. Reductions in gradients along major streams that traverse subsiding basins have increased the flood hazards in the central and lower parts of the lower Santa Cruz basin.

Continued ground-water depletion will result in continued land subsidence. Enlargement and extension of existing earth fissures and formation of new fissures can be expected. Damage to engineering structures by differential land subsidence and (or) earth fissures can be anticipated. The potential for land subsidence and earth-fissure damage is an important consideration in the design of major engineering structures in subsiding areas.

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LAND SUBSIDENCE IN A CALIFORNIA COUNTY: THE LEGAL ISSUES

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Abstract

In the early years of this century subsidence of land surface in Santa Clara County occurred by reason of intensive pumping of groundwater for irrigation. Under California law, there is no effective State control of groundwater extraction. There are, however, private rights in groundwater as a common resource. A person injured by overpumping may sue upon an allegation that the pumper is taking more than his share -- the right to groundwater being not a right to a stated amount but rather to a share of the available supply. In addition, a local public agency having the power to tax the activity of withdrawing groundwater can by this means exert a measure of control. In the absence of statewide regulation, local public agencies have been moving to secure regulatory powers which include direct control as well as taxation of groundwater extractions.

Land Subsidence in Santa Clara County

The Santa Clara Valley is the extension southerly and easterly of San Francisco Bay. The south arm of the Bay is simply a drowned valley containing the former thread of Coyote Creek, the area's major watercourse. The valley is drained by a number of smaller creeks rising in the foothills to the east and west. These have in the course of geologic time shifted about and laid down their silts and gravels so that under the valley today there are many lenses, layers, and fingers of material coarse enough to store water readily. In the north part of the valley these aquifers all incline toward the Bay, in the south toward Pajaro River which is tributary to Monterey Bay and the Pacific Ocean. As the old rivers flattened they dropped progressively finer and finer materials. Thus, we have receptive gravels on or close to the surface at the base of the surrounding hills; near the Bay we have tight clays overlying these water bearing bodies.

In the Spanish and Mexican colonial period there was no substantial use of groundwater. Dry farming and raising cattle for hides was the rule; there was very little ditch use.

After statehood, more intensive farming began. However, not until about the turn of the century with the invention of the deep well turbine pump, did farmers begin their reliance on wells and begin creating the problem of overdraft and the necessity for groundwater management.

By 1913 the lowering of the valley's water table had become noticeable and a few leaders began the long effort to reverse it. After the Great War, which had preoccupied everyone, efforts were made to organize a public agency or "district" to do something about the problem.

A word of explanation: "Districts", as the term is used in California, are a form of local public agency created by the State Legislature to perform specialized services and exercise limited powers within a described area. They have specified sources of revenue; (taxes and assessments or charges) and they are self-governing by an elected body. They are, in effect, public corporations.

By 1929 the conservative and hardheaded majority of the prune and apricot growers of the Santa Clara Valley had been convinced by a series of experiments that you could actually put water underground and that it was vital to them to do so. Artificial recharge had been proposed in detail in a report made in 1921 by a pair of engineers hired by a local association. But until things got really bad, the means were not thought practical or credible, or if credible were seen as wildly expensive.

The means are entirely familiar today and consist of catching winter flood flows in conservation dams and releasing them in the summer to the gravels at the base of the hills, the entrance to the aquifers. Percolation movement is then governed by gravity and the character of the receiving strata.

The new District, formed in 1929, built a number of such dams and began the process of artificial recharge of the underlying basins. The process included, by necessity, scheduled releases or "pass throughs" of flows sufficient to satisfy vested rights downstream. Percolation in the stream beds was encouraged by low check dams bulldozed up in the spring and washed out in the fall. It also found, bought and developed off-stream pervious areas for use as percolation ponds. Riparian owners were allowed to buy a supply of water thus appropriated and released. This initiated a principle we are still working on, namely, that the basin is directly assisted by any substitution of surface water for a pumped supply.

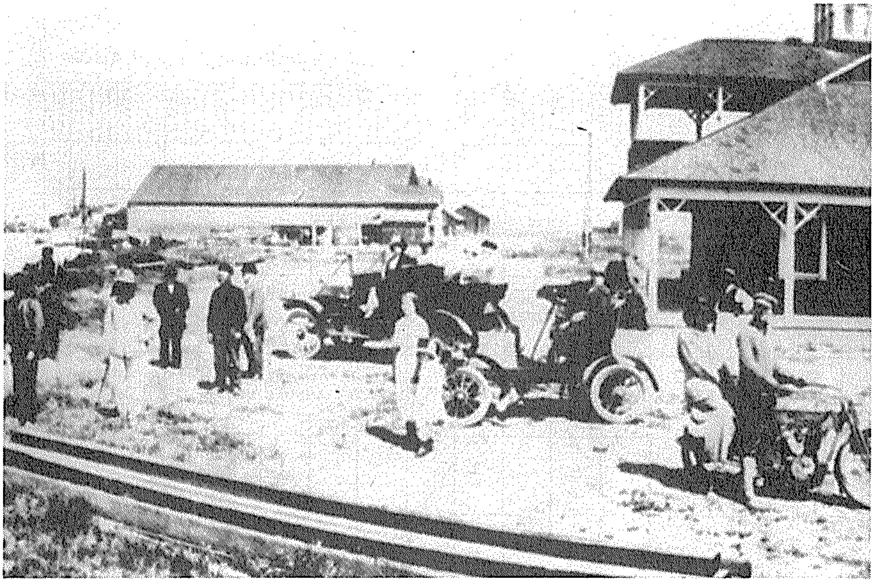
This local program worked spectacularly well. By 1942 the well levels moved up. Artesian flows in the bayward pressure zone were restored. However, under intensified use of water during World War II and thereafter -- everyone in the country seemed to be moving to Santa Clara County -- the depth to water again lengthened and a condition of overdraft became chronic and growing.

In these circumstances it was decided that water must be imported. Some of our cities lying along the rim of the Bay have access to the City of San Francisco's "Hetch Hetchy" line which brings good water from the Sierra Nevada. They began buying some and to the extent they did and do they leave water underground for others who cannot buy from this source. The federal statute, called the Raker Act, which permitted the City to store water in a national park forbids sales for resale or for irrigation and also limits uses to domestic or municipal. Hence, this supply cannot be put underground since a major use of water from wells is agricultural and since groundwater is pumped for commercial sale.

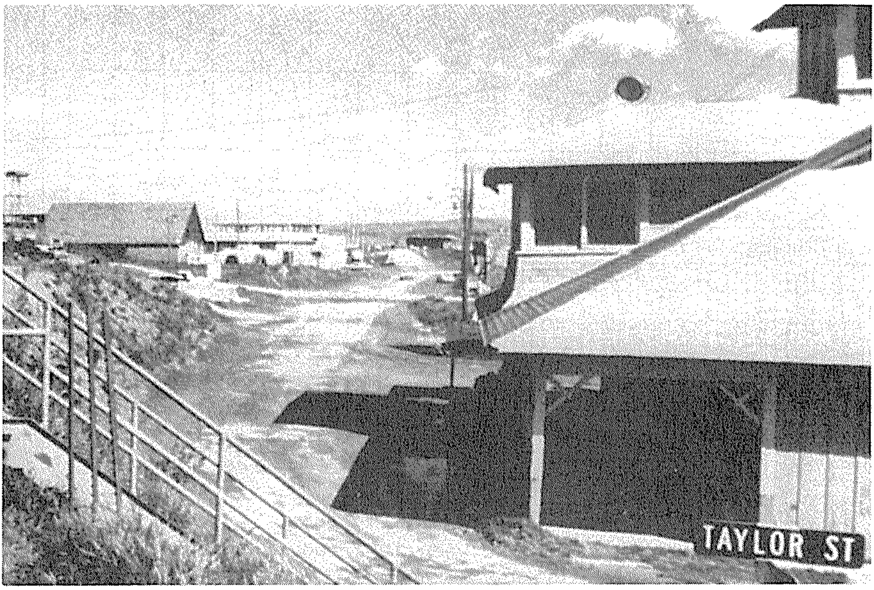
The California State Water Project provides another source. We import about 100,000 acre-feet a year, by purchase from the state. This water we can percolate or treat and serve wholesale. We do both. In either event we are conserving water underground.

A third source will be, eventually, a unit (the San Felipe Division) of the Federal Central Valley Project. This is expected about 1987 and, again, we can serve it on the surface or place it underground for pumpers to use, or both.

A falling water table with consequent dry wells or expensive well deepening is not the only nor perhaps the worst result of long continued overdraft of groundwater. Land surface subsidence in northern Santa Clara County has occurred over an area of some 622 km^2. Loss of elevation in downtown San Jose, the principle city, is 4 to 4.25 meters. Lands directly littoral to San Francisco Bay have sunk as much as three meters. The result for the people attempting to live or do business on Bay front property can be imagined. A dramatic illustration of the condition will be seen in the pair of contrasting photographs shown below.



SOUTH BAY YACHT CLUB - 1914
San Francisco Bay is to the left



SOUTH BAY YACHT CLUB - Today
Note 3.66 meter high levee on the left

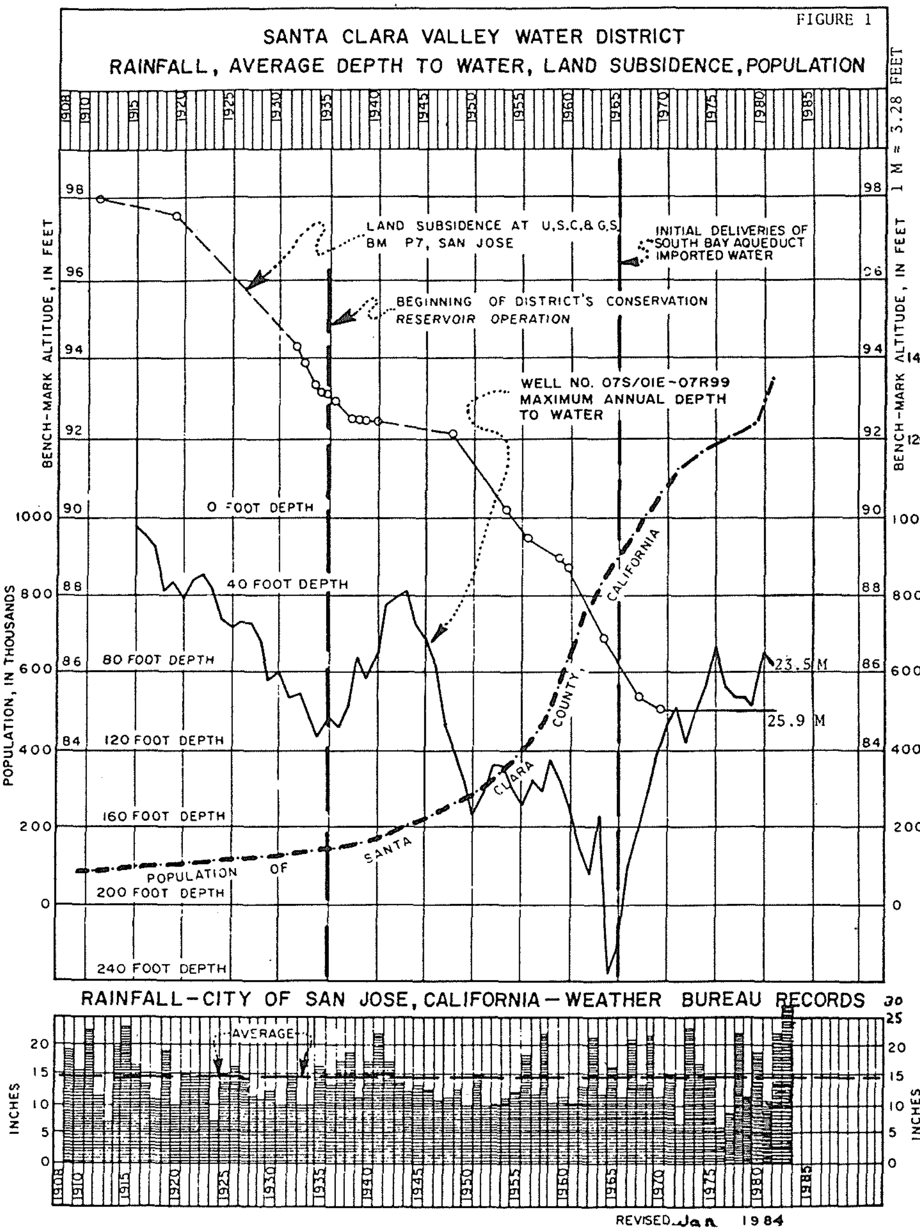


FIGURE 1

Surface subsidence took place between 1912 and 1969 when it was brought under control. A graphic representation of the course of the phenomenon is shown as Figure 1. A correspondence between a program of artificial recharge of the underground and control of subsidence is apparent. Subsidence in this Valley was carefully observed and measured by the

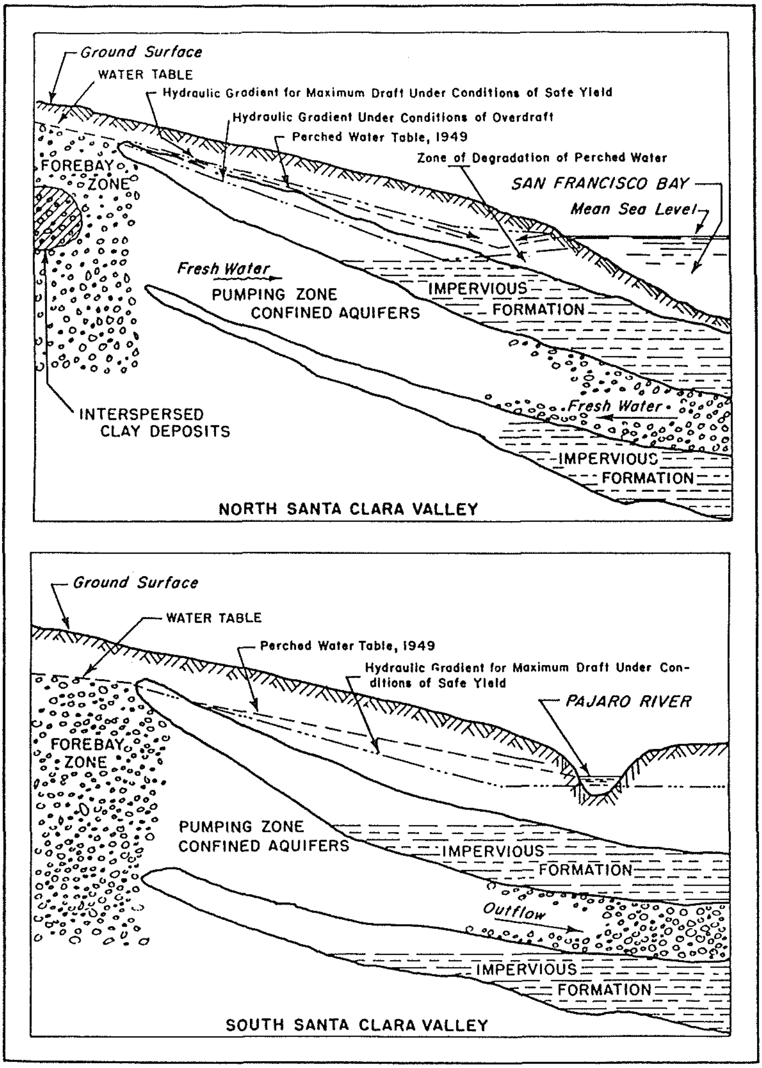


Figure 2. Cross Section of Ground Water Reservoir

Source: California Department of Public Works, Division of Water Resources, Santa Clara Valley Investigation. (Sacramento: State Print Off., June, 1955) (Water Resources Board Bulletin No. 7.)

United States Geological Survey. It is an interesting fact that it was the first in the United States to be recognized as having been caused by overdrafting the groundwater supply. See Poland (1978): reference should be made to this thorough investigation and report for a technical description of the process. In general it appears to be established that surface subsidence may follow, and in the Santa Clara Valley did follow the loss of pressure within confined aquifers by reason of over pumping.

The loss caused compression of the lenses made up of clays or other fines in such bodies. The clay cap, no longer supported, subsides under the influence of gravity. Unhappily the loss of elevation is permanent, no feasible method of reversing the process is known. What can be done is to recharge the gravelly portions of the water bearing bodies, the "ground water reservoir" by percolation in the elevated forebay zone, thus halting the compression process. See Figure 2. Manifestly the means of doing so include artificial introduction of groundwater to augment the natural effect of rainfall as well as lessening by every means the rate of withdrawal. In the Santa Clara Valley we are doing both through the use of a local public agency, the Santa Clara Valley Water District. In what follows we will examine the law governing groundwater and consequently the law governing land surface subsidence control.

Groundwater Law

The subject greatly exceeds, of course, the limits of this paper. We can do no more than briefly outline the historical basis of the law and its modern applications. We must be aware generally of rights to groundwater if we are to discuss its management with the objective of controlling land subsidence. We must ask who, if anyone, "owns" percolating groundwater.

The settled eastern shore of what became the United States began and continued for many generations as a British colony. Along with much else, we took our governing law from that country. When California was ceded by Mexico under the 1848 Treaty of Guadalupe Hidalgo and in due course was admitted as a state, the English Common Law was offically adopted as our basic rule in the absence of contrary statute. The Common Law is the product of a long history of individual decision making; basically it is judge and custom made. Lawyers attempting to predict what a court will do search out precedents, prior examples of what courts have done. Thus to find the law governing rights to groundwater, 19th Century English cases were consulted. These determined that, as an owner of land owns to the center of the earth, groundwater found in that theoretical prism is a part of that absolute proprietorship. The result was that a person injured by his neighbor who was over-pumping the common underground reservoir, or "basin", had no recourse in law.

In the 19th Century the judges and litigants had no understanding of groundwater and the notion of its absolute ownership was largely a confession of that ignorance. The California Supreme Court, better instructed in hydrogeology, has decreed a different rule. See Katz v. Walkinshaw (1903). The law now recognizes a groundwater basin as a common resource; that it is subject to abuse and depletion and that the injury occasioned by abuse should have a remedy. It is recognized that water moving on the surface or, more slowly, underground cannot be "owned" by any one. The right to water is a right to use it, after which the hydrological cycle continues. Rights to use the content of a groundwater basin are, under our law, shared among the owners of the surface. The primary right is held by an owner for use upon his overlying land. If there is any surplus others may pump and use the supply. If there is a shortage, these others, call "appropriators", are cut off first and among themselves, the rule requires that those last to begin pumping are cut off first. "First in time is first in right". Overlying owners have no priority based on time. If after appropriators stop withdrawing there is still a shortage, overlying owners must share the inadequate supply. Hence, the right of the overlying owner is called "correlative".

Those are the simple, straightforward rules. Unfortunately in a fact situation of any complexity they cannot be followed. The underground is a major source of supply in a country where surface streams are scarce, intermittent and unreliable. The uses of appropriators cannot be ended in that order. In a recent landmark decision of the California Supreme Court, Los Angeles v. San Fernando (1975), it was conceded that iron mathematical rules would have to yield to the inescapable fact that a major municipal supply could not simply be ended as practical matter; that equitable divisions outside the theory would have to be made in many cases.

Legal Issues

Since 1914 anyone in California wishing to take and use surface water who is not acting upon a riparian right (or other right established prior to 1914) must seek a permit from the State. Unlike most other western states, California has never extended this permit system to groundwater. Recently a Commission to Review California Water Rights Law was established by the Governor. After much study and extensive public hearings the Commission released its final report. The following statements are taken from the Summary (1978).

> "There are several types of problems related to overpumping, besides the problem that the groundwater supply in a given area may be exhausted. One example is seawater intrusion into fresh water acquifers, which occurs when groundwater extraction increases to the point that normal seaward movement of fresh water is decreased and seawater moves inland. Subsidence also results from overdraft.
>
> To deal with existing groundwater problems, the Commission recommends that legislation on groundwater management, the adjudication of groundwater rights, and conjunctive use of surface water and groundwater resources be enacted. Because of the various levels and types of existing management programs and the substantial differences in groundwater basin conditions and needs in the state, the proposed legislation allows for flexibility whenever possible.
>
> The basic premise of the Commission's proposed legislation is that local management if it is properly undertaken, offers the best opportunity for workable and effective control. Local entities should be given the primary responsibility and necessary powers to develop and implement management programs. The proposed legislation anticipates that areas that are already well-managed or that do not have critical groundwater problems will be "inactive", that is, they will not be required to have a designated groundwater management authority or a groundwater management program. Such areas, however, may choose to have the inactive classification removed in order, for example, to obtain the powers granted to groundwater management authorities. The Commission intends that the requirements of the proposed legislation not jeopardize working management efforts or require any unnecessary management actions."

The Commission suggested enactment of a statute to provide that local entities or districts in areas of critical overdraft be empowered to develop a groundwater management program for the area and perform groundwater management. If there were no suitable local entity, one was to be formed to be called a "Groundwater Management District". The necessary statute was never enacted principally because the law was to require that local programs be subject to state review. Agriculturalists, especially those in the great San Joaquin Valley -- an area of critical overdraft -- feared to permit any possibility of loss of local powers of decision over withdrawals.

In the absence of statewide legislation some local agencies have drafted and secured enactment of authority to management groundwater within their own boundaries. These are examples of such provisions:

Basin Equity Assessments. A governing board determines for each oncoming year the amount of groundwater that can be safely withdrawn from the basin. Each pumper is then allocated a percentage of that yield (based on past extraction records). A pumper who takes more than his allocation pays an assessment. A pumper who takes less receives a payment upon the same basis.

Withdrawal Permits. No one can extract groundwater without a permit from the district and the permit may be withheld or limited upon a finding that the basin is overdrawn. The district may also require that wells be located and properly spaced apart.

Groundwater Extraction Charges. An excise is laid upon the activity of extracting groundwater measured by amount withdrawn.

Direct Charges. The district may impose a charge upon each unit of land overlying a basin to support the district's groundwater management activity. This is not an ad valorem tax but is more in the nature of an assessment based upon the benefit conferred upon the ownerships by reason of management of their underground resource.

Where a local political body and its officers and employees have acquired power to regulate, the regulatory function becomes a matter of daily decisions, of judgment. It would be essential that the making of such decisions be protected from civil suit in order that the system can work. Thus, for example, a decision to deny a well permit to an orchardist might cause the death of his trees. If the decision was made without fraud or collusion, the agency should be immune from an action to recover damages. Such immunity is provided in California by general statutes insulating government from tort claims (with specified exceptions not pertinent here).

Private parties can and do sue each other upon an allegation that more than a legal share is being taken. The plaintiff can describe his harm, which may include subsidence of his land surface, and demand money damages or, more usually, an injunction limiting the extractions to a proper figure. We have seen that under California groundwater law the rights of overlying owners are correlative. However, there are crushing burdens on a litigant in such a suit when more than a very few pumpers are involved. The reason is that before it can be said that pumper A is taking more than his share we must find what pumper A's share is. In theory, at least, each overlying owner has a right to use water from the basin on his land and he may begin that use at any time. He does not lose his right by non-use or delay. The limits are that he cannot waste it and he must share a shortage. When may we say there is a shortage? When the safe yield is being exceeded. What is the safe yield? It has been defined as the maximum quantity of groundwater that can be withdrawn annually under a given set of conditions without resulting in a lowering of the groundwater level and eventual depletion of the supply. But this definition must be subject to the right to use the basin for storage through wet and dry periods; the right to fill it in the one and to drain it down in order to ride through the other. Thus there may be temporary intentional surpluses and lawful temporary deficiencies. Los Angeles v. San Fernando (1975).

If we can agree upon a safe yield, that yield is to be divided not upon the basis of use historically established but upon a basis of reasonable need. And all of this assumes that mere appropriators have been cut off -- a practical and political impossibility in most cases.

Such a legal process is called a groundwater basin adjudication and it will perhaps never again be attempted. The usual pattern today is professional neutral research which bases an agreement, application to a court to approve the agreement and installation of a court appointed "water master" to enforce it.

Groundwater Management in Santa Clara County

Local Management of Water -- surface, subsurface and flood -- is in the hands of Santa Clara Valley Water District. This is an example of the agencies I have mentioned, one created by the State Legislature to perform specified services within a described area. The District does not enjoy most of the useful powers which would permit direct control of the resource.

As I have stated, the District captures winter flood flows and percolates the supply into the underground. It imports large amounts of water and much of such water is also percolated. How can this constitute management? We seem to be devoting effort to replenishment of a groundwater supply and then abandoning it to the individual choice of every pumper.

As we lack the authority to control extractions by direct order, we must do it indirectly by suasion, by a system of rewards and punishment. The first tool to this purpose is the groundwater extraction charge. Legislation amending the District's governing act to permit the charge was adopted in 1962.

The groundwater extraction charge, or "pump tax" as it's often called, is in legal contemplation no more than a familiar excise. It lays a charge upon the activity of extracting groundwater and measures the charge by the amount extracted. It is not a sale of groundwater -- after all the overlying owner is presumed to have a right to take and use the percolating water beneath him. If you would nudge people into using surface water in order to preserve the underground for periods of surface water shortage, you will want to insure that groundwater is not measurably cheaper to use.

This is economic urging, often the best kind. Another form of such urging we have used is tax rebate. The legislative authority to do this was added to the District's Act at our request. Thus, an entity such as a city which imports water is rewarded by a rebate of a portion of the property taxes collected in its area upon the theory that that area does not benefit from groundwater replenishment as much as others which rely more heavily on that source. By the same token that area has, by importing, left more water underground.

The benevolent and beneficent manipulations by which we manage groundwater in the Santa Clara Valley depend on a decision agreed to by all concerned. This is called the Pricing Policy. The District, as I have indicated, does not merely restore and maintain the groundwater basins it overlies, it also buys, treats and wholesales massive quantities of surface water. We set the price of that surface water. The basic element of the pricing policy is the determination to view all water without regard to source (and hence without regard to varying costs) as pooled, as one supply. If we did not do that, if we related the price of water to its costs, we would find it impossible to get people to choose a more expensive supply, even though such a choice is greatly in their long range interest.

The thing is done with what could be called an "equalizer". We take the groundwater extraction charge (which is set each year by the Directors) as a basic user charge. To that figure we add a variable called the "treated water surcharge" and the total is made the cost to our wholesale customers of our potable water. The treated water surcharge is intended to produce a sum which is roughly equivalent to the per acre-foot cost to a pumper imposed by the groundwater extraction charge plus all his other costs of extraction, capitalized well, pump, energy and all. In such a situation he does not easily prefer pumping. He is prepared to switch to a surface supply as long as it is available and to rely on the basin storage for periods when surface supplies are short. And that is, of course, the idea.

You'll have seen that our groundwater management program represents a choice made. Presumably we could have let the condition of overdraft continue and worsen until only those with the deepest wells survived or until, as some economists urge, water is reserved by scarcity to its highest use. We could have instituted an adjudication and established all the relative rights. The first was politically impossible; the second takes too long and costs too much and would hve ended by sharing out a shortage. The methods we use are improvisations to approximate the control which we lack the power to impose directly. They work only because an imported supply can be offered in substitution.

In summary then, the Santa Clara Valley Water District is engaged in management of its basin (actually three interconnected basins). The process is indirect but roughly appropriate and effective: it depends on an ability to price water to the user in such a way as to induce conjunctive use and preservation of the groundwater resource. I have given examples above of more direct methods. We may be seeking and using them in the future. But as of today the process has worked and the result has been an end to land surface subsidence in Santa Clara County.

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A DETAILED CASE HISTORY: MARKERWAARD, NETHERLANDS: SOME INTRODUCTORY REMARK

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Abstract

The story of the Markerwaard takes place in the north-western part of the Netherlands. In the beginning of our era a fresh-water lake, Lake Flevo, was present there. During heavy storms the North Sea extended its influence in the direction of this lake and in the 12th century the ZuijderZee came into being. However, it wasn't until the middle of the 16th century that the ZuijderZee attained its final shape.

Probably the oldest plan to recover these lost lands dates from 1667, but of course this plan could not be executed at that time. Later on, in the second half of the 19th century, several plans for reclamation were published, but none came into execution. Due to a heavy storm in 1916, a large part of the province of North Holland was flooded and Amsterdam, the capital of the Netherlands, was endangered. That was the direct motive for a bill dealing with the reclamation of the ZuijderZee, which was offered to the parliament. This bill was accepted and the ZuijderZee (Reclamation) Act was published in 1918. The project, described in this act, consisted of:

(a) the construction of a 2.5 km enclosure dam from the coast of North Holland to the island of Wieringen

(b) the construction of a 30 km enclosure dam from the island of Wieringen to the coast of Friesland

(c) the reclamation of 4 polders (purpose: increase of the agricultural area, which was then, the years after the first World War, important for food supply).

The enclosure of the ZuijderZee had several advantages:

(a) a shorter coastline, so better protection against flooding,

(b) the formation of a fresh-water reservoir, important for the water supply to the surrounding agricultural areas in the dry season,

(c) more favourable drainage of the surrounding areas and of the new polders,

(d) a more simple and more economical building of the dikes around the polders to be reclaimed.

The originally salt-water reservoir, caused by the construction of the enclosure dam gradually would change into a fresh-water reservoir, due to the supply of fresh-water by the river IJssel. This necessitated the building of two sets of sluices in the enclosure dam.

After a period of measurements and calculations the construction of the enclosure dam and of the dike around the Wieringermeer started in 1927. The Wieringermeer, the first polder, was reclaimed in 1930. The execution of the enclosure dam lasted until the 28th of May 1932, when at 13.02 o'clock (two minutes later than scheduled) the last gap was closed. The ZuijderZee ended its life and Lake IJssel began its life. Between brackets, last year, 1983, the enclosure dam was honoured by the American Society of Civil Engineers as an International Historic Civil Engineering Landmark.

The activities related to the North-East Polder started in 1937, the polder was reclaimed in 1942. The bottom of Lake IJssel consists of soft layers, which could cause difficulties in the construction of the polder dikes. So part of this layer was dredged and replaced by sand; the lower

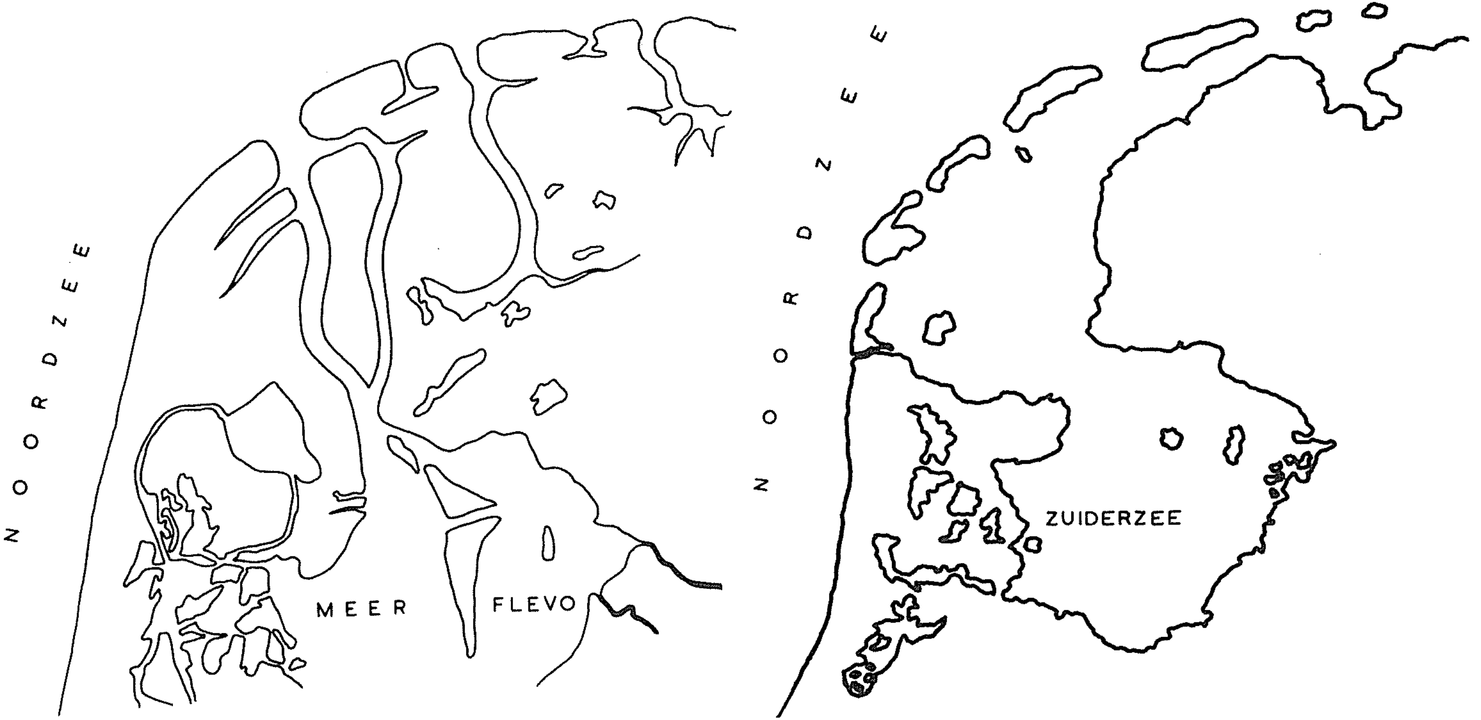

FIG. 1--The Netherlands in the 1st century (left) and in the 16th century (right).

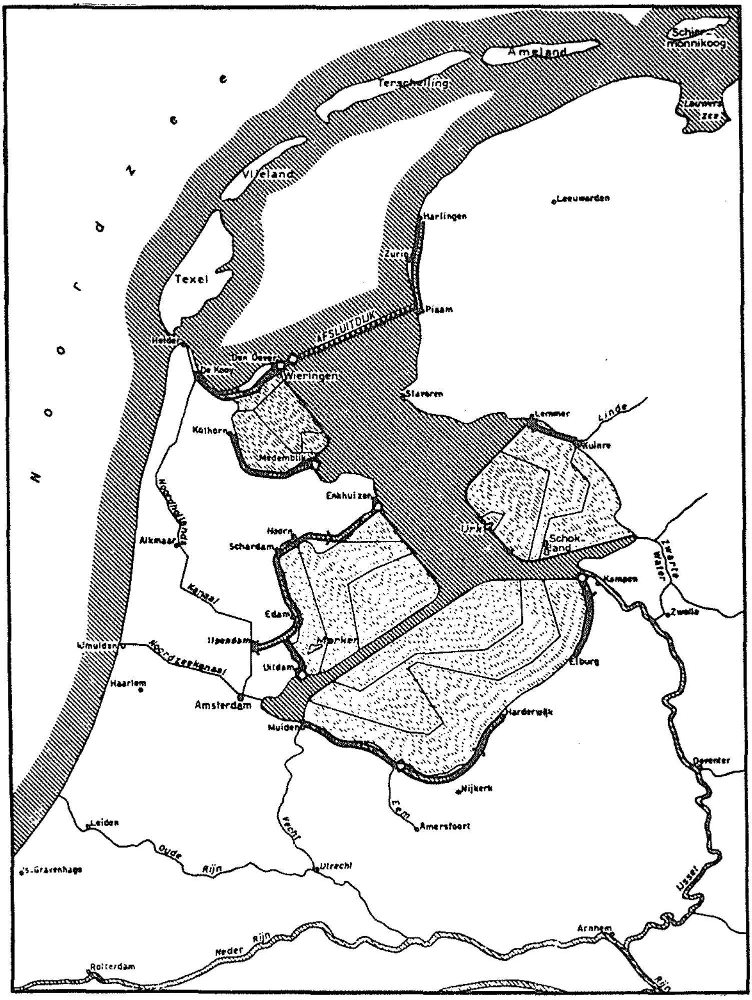

FIG. 2--Adopted plan for enclosure and reclamation of the ZuijderZee.

meter of the soft layers was maintained however to prevent seepage underneath the dike from the lake to the polders. This North-East Polder directly joins the "old land" and it was observed that the reclamation and the following lowering of the groundwater level caused some damage to the agriculture.

The third polder, projected in this south-eastern part of Lake IJssel, is reclaimed in two stages, Eastern and Southern Flevoland. The dike building of these polders started respectively in 1950 and in 1959; the reclamation was completed in 1957 and in 1968. Between the dikes and the "old land" now border lakes of various widths were left to prevent influence of the reclamation on the groundwater levels in the "old land." In the meantime the ideas about the utilisation of the new polders had changed. In the Flevoland polders large areas are reserved for urban developments, for recreation and for nature conservation.

Finally the last polder, the Markerwaard, projected in the south-western part of Lake IJssel, is waiting for reclamation.....or not. During the last years the environment became more and more important and so it is not surprising that some people raised their voices against the reclamation. They emphasize the importance which the area for the present has in relation to recreation, wintering water-birds, the fishing industry, among other things, and they point to the adverse consequences of the reclamation to the situation on the "old land." All this led the government to reconsider the reclamation of the Markerwaard and consequently a comprehensive study was started. An important part of this study was devoted to the consequences of the reclamation and the subsequent lowering of the groundwater level on the buildings and agriculture at the "old land" and to possible measures to prevent damage entirely or partially. The lectures of this session will review the results of the last mentioned part of the study.

AN INTEGRATED STUDY TO FORECAST AND TO PREVENT DETRIMENTAL EFFECTS IN THE PROVINCE OF NORTH HOLLAND RESULTING FROM A CHANGE IN THE GROUNDWATER REGIME AFTER THE RECLAMATION OF THE MARKERWAARD POLDER.

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Abstract.

The reclamation of the new Markerwaard polder may have detrimental effects on the eastern part of the province of North Holland as a result of changes in the groundwater regime in the area. An integrated study was organised to examine possible compensatory measures.

One of the most important goals of the study was the development of a methodology to forecast the geohydrological changes in the area, their influence on magnitude and rate of settlement of the subsoil and the probable damage caused by these geotechnical changes to buildings and infrastructure of the area. A method was furthermore developed to compute the lay-out and the scale of magnitude of artificial groundwater recharge works to compensate, fully or partially for land subsidence in those parts of North Holland most susceptible to detrimental effects.

1. Introduction.

1.1. General

The reclamation of the Markerwaard polder will probably be the last phase of the IJsselmeer development project. (fig. 1) This project started in 1930 with the enclosure of the former Zuyderzee, now Lake IJssel, and in that same year with the reclamation of the Wieringermeer polder (area: 200 km^2). In 1942 the North Eastern Polder was reclaimed (480 km^2), in 1957 the Eastern Flevoland polder (540 km^2) and in 1968 the Southern Flevoland polder (440 km^2).

The study area lies in the north western part of the Netherlands (figure 1). The Markerwaard polder, covering an area of 410 km^2, is to be reclaimed in the south western part of the Lake IJssel. The phreatic level in the polder will drop some 5 to 6 metres below the present level, which will induce a considerable flow of groundwater into the polder and thus cause a substantial change in the groundwater regime in the adjacent peripheral lakes, other recently reclaimed areas and the older polder land of the province of North Holland.

The study is focused on the changes in the groundwater regime of the province of North Holland, as it is there that various types of damage may occur as a result of changes in the piezometric head of the groundwater in the shallow aquifers. Research conducted by the Public Works Department in 1976 showed that the main geohydrological changes would occur in the boxed section of

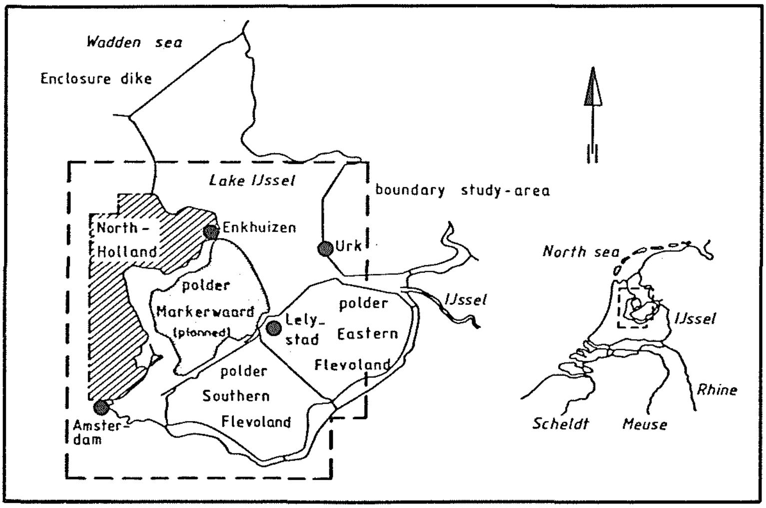

figure 1 Study area

figure 1. An area of 500 km^2 in North Holland (the hatched section) was considered most likely to be affected by the predicted changes.

1.2. The problem and the aim of the study

The following harmful effects may occur in North Holland as a result of a change in the groundwater regime in the Markerwaard polder and its surroundings:

- damage to agriculture, in the form of lower crop yields, caused by a drop of the phreatic level and increasing soil moisture deficiency in the growing season;
- damage to animal and plant life through changes in the abiotic conditions resulting from land subsidence and a drop in the phreatic level in the growing season;
- damage to buildings and physical infrastructure through settlement of the Holocene compressible deposits.

The study was designed to determine the extent of the various negative effects and their spatial pattern, the time span over which they would occur, the locations where damage could be prevented and the compensatory measures (e.g. recharge wells and infiltration grooves in the peripheral lakes) required to achieve this. Probable positive effects as a result of the reclamation are not investigated.

A feasibility study of compensatory measures was carried out. It was not restricted to questions of technical feasibility but also looked at economic viability by comparing the cost of such measures with the cost of damage. Consequently, the hydrogeological and geotechnical aspects were studied together with the agrihydrological, ecological and civil engineering aspects, thus enabling a forecast to be made of the damage to buildings and objects in urban areas, to agriculture and animal and plant life on the basis of the following:

1. The calculated drawdown of the piezometric levels in the Pleistocene aquifers and drop in the phreatic level;
2. The settlement of the Holocene compressible deposits;

3. The additional negative skin friction on pile foundations.

Use of the same groundwater model in combination with the existing calculations concerning the rate of drawdown of the piezometric levels has shown where and how much water has to be injected into the aquifers to prevent or reduce drawdown of the piezometric level in those areas vulnerable to damage. The costs of various compensatory measures have also been estimated.

1.3. Background to and reasons for this study

Hitherto, there have been no systematic inquiries into damages caused by settlement in urban areas, nor any quantitative study to forecast possible damage.

Further reasons for this study were:

- the Government has not yet decided whether or not to reclaim the Markerwaard polder. A fresh round of public participation and consultation was launched by the Government in 1981 with the object of reconsidering all relevant aspects so as to arrive at a balanced judgment;
- numerous new data have been obtained on the hydrogeological and geotechnical condition of the area since 1970;
- various numerical calculation models facilitating extensive, detailed and rapid calculations have recently become available.

  It was therefore possible to approach this problem on a more precise and comprehensive basis.

2. Structure of the integrated study.

The main objective of the study was to obtain a spatial picture of the order of magnitude of the various types of damage that could result from changes -both spatial and temporal- in the piezometric head on the first Pleistocene aquifer in North Holland. Also investigated were the technical and financial aspects of taking compensatory action. This enabled possible damage to be weighed against the cost and effort of preventing it and this information could then be used when deciding whether or not to reclaim the Markerwaard polder.

Since the study embraced various disciplines, it was divided into seven sections, each dealing with the scale of spatial and temporal changes in the factor which, in that particular discipline are associated with the occurrence of specific types of damage in the study area as a result of the reclamation of the Markerwaard polder. Figure 2, a diagram of the interrelationship of these various sectoral studies, illustrates their interdependence,viz. the extent to which the results of one may influence those of others.

The various sectoral studies were started concurrently, with the exception of the geological and geohydrological studies, which had been started earlier.

They were conducted by six organisations, which worked together on the project under the supervision of the Public Works Deparment, Rijkswaterstaat, Directorate of Water Management and Hydraulic Research, Northern District. The following calculation models were used:

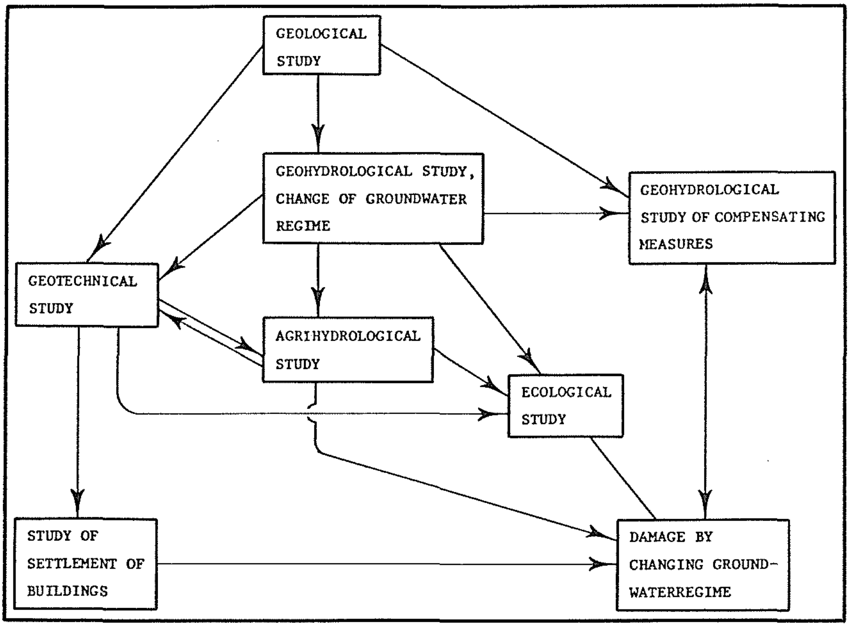

figure 2 Correlations between different study aspects.

- a steady saturated groundwater flow model FIESTA based on the finite element method (Rijkswaterstaat, 1978) of an area of more than 5000 km^2 (figure 1) to determine the drawdown of the piezometric levels in three aquifers.
- two numerical groundwater models: MOTGRO and Rijtema/de Laat, for the unsteady unsaturated flow, to calculate the drop of the phreatic level and the probable crop reduction.
- several soil mechanics models to determine: the settlement of the compressible Holocene deposits after 30 years; the speed of settlements for 30 specific soil types in the North Holland study area; and to calculate the drop in pile foundations through additional negative skin friction, caused by settlement of clay and peat deposits.
- a simple axial symmetric unsteady saturated groundwater flow model of a semi-confined groundwater system to calculate the velocity of the piezometric decline under and around the new polder, for different reclamation times of the polder, and different assumed hydrogeological- and consolidation constants.

The basic premises of the integrated study and the major features of its design are discussed briefly below. For more information on these points in each separate study, see the relevant papers (Vos et al 1984, Westerhoff et al 1984, Hannink et al 1984, Carree et al 1984, van Bruchem 1984, Barends 1984).

1. The study is based on the superposition principle, which means that the geohydrological aspects of the reclamation of the polder are superimposed on the present situation.

The damage is estimated on the basis of the resultant changes. It is postulated that, in the past, the piezometric head of the groundwater has not been lower than now.
Geohydrological and geotechnical changes that occured in the past have been incorporated in the present hydrological and geotechnical situation.

2. The damage is expressed as costs entailed in repairing damage to buildings etc., or preventing damage to agriculture by, for example, additional sprinkler systems. Damage to plant and animal life cannot be expressed in monetary terms. It is thought that populations of rare plants, animals and birds could be adversely affected.
3. The finite elements model of the steady saturated groundwater flow of the study area is calibrated in relation to the present (1978) piezometric surfaces (3 aquifers) and seepage and drainage data of polder areas (I.C.W., 1982). The model is also partially calibrated on the basis of the geohydrological changes resulting from the reclamation of Southern Flevoland in 1968.
The expected speed of the piezometric drawdown of the groundwater level in the aquifers, caused by the reclamation of the Markerwaard polder, is based on a series of measurements made, both in and near Southern Flevoland after the reclamation of this polder and on some simple indicative calculations.
4. It is assumed that compensatory measures, such as the use of recharge wells along the coast of North Holland or the provision of infiltration grooves in the peripheral lakes, are technically feasible (blockage can be prevented).
5. The cost and scale of works which will totally or partially offset the damage are investigated.
6. Attempts were made to determine the extent of damage costs and cost of compensatory measures.

3. Main results and conclusions.

1. The detailed results of the sectoral studies, set out in the papers referred to above, show that the study achieved its goal of presenting a spatial picture of the various possible types of damage and specific compensatory measures considered feasible.
2. If no countermeasures are taken damage to buildings and infrastructure as a result of settlement and additional negative skin friction on (wooden) pile foundations is expected to be considerable.
A spatial pattern of the extent of this type of damage was obtained with a damage calculation model.
The results of this calculation model are less reliable than the results of the models of the other study aspects, owing to a lack of knowledge of and experience with the relationship between land subsidence, rate of settlement, damage to buildings and restoration damage. So the calculated amounts of damage to buildings and infrastructure are no more than indicative.
3. Harmful effects on plant and animal life as a result of a change in the groundwater-regime are expected to be nonexistent or negligible compared to the effects of land subsidence mentioned above.
4. Countermeasures to prevent land subsidence and so damage to

buildings and infrastructure in North Holland are feasible. Their costs are less than the expected damage.

5. Use of recharge wells, located along the east coast of North Holland or clustered near urban regions most vulnerable to damage, is more effective and gives more operational flexibility than use of infiltration grooves, dredged in the peripheral lake.
6. It has been established that damage is proportionate to the extent of the countermeasures, more exactly, to the amount of infiltration water - both in the case of recharge wells and infiltration grooves.
7. Field experiments on a semi-technical scale have to show, in more detail, the operational and technical aspects of feasible countermeasures in order to come to an optimal choice.
8. This research method yielded useful information on the correlation between the aspects investigated.
   The integrated calculation method developed makes a rapid comparison of a wide variety of variants with divers basic premises possible and enables policy makers, among others, to choose optimum compensatory measures and their location.

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QUATERNARY GEOLOGICAL FRAMEWORK OF NORTH-HOLLAND AND THE MARKERMEER (THE NETHERLANDS)

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Abstract

Quaternary geological data are presented as a basis for the extensive geohydrological and geotechnical studies performed to investigate possible harmful effects of the Markerwaard reclamation project.

1. Introduction

A sound investigation on the impact of the proposed reclamation of the Markermeer on the adjacent land area of North-Holland must be based on a thorough knowledge of the composition of the subsurface of the entire region.

For this reason, the Geological Survey of The Netherlands was asked to present a detailed geological framework of the study area. This framework has been translated into a geohydrological model by IWACO BV. Modifications of the ground water regime will have influence in the topmost 250 - 300 metres of the earth's crust, consisting of poorly consolidated clastic deposits saturated with brackish and saline water. Lithostratigraphy and chronostratigraphy of the Pleistocene deposits, as described in section 2, is based on a large number of deeper borings and is derived predominantly from the comprehensive study on the Pleistocene Geology of North-Holland by Breeuwer & Jelgersma (1979).

The geological model of the Holocene (section 3) is the result of a detailed study of thousands of shallow borings and cone penetration tests. This model served as basis for geotechnical studies, done by the Delft Soil Mechanics Laboratory and the Heidemij Nederland BV.

The composition of the Holocene coverbeds is expressed in a lithological succession legend (see section 4). On this basis, the land area surrounding the proposed polder was divided into subareas displaying certain soil mechanical and hydrological characteristics in relation to possible future changes in the ground water regime.

2. Pleistocene

Geologically, The Netherlands forms part of the subsiding basin of the North Sea, which is filled with a poorly to unconsolidated sediment succession. The thickness of these sediments generally increases in NW direction (Van Staalduinen et al., 1979). The top of the consolidated hardrock consists of Upper Cretaceous limestones and is situated at a depth of 900 - 1000 metres below the surface in the study area. The Cenozoic deposits were not affected by substantial folding, and tectonic structures are confined to block faulting in deeper strata, which led to NW-SE oriented horsts and grabens.

Coarse-grained clastic deposits such as sands and gravels, may serve as aquifers, whereas clay and peat beds are to be considered impermeable (aquicludes). The predominantly clayey Tertiary deposits may attain a thickness of more than 500 metres. The top of the Tertiary is formed by a 20 - 50 metres thick marine-clay bed of Pliocene Age, belonging to the Oosterhout Formation (no. 1 in Fig. 1).

The overlying Maassluis Formation (no. 2 in Fig. 1) is of Early Pleistocene Age and consists of shell-bearing sandy clays and fine-grained sands. The topmost part of this shallow marine formation usually contains clay beds. During the Tiglian (see Fig. 2) the sea retreated from The Netherlands and the study area became part of the continent for a long period of geological time. In this area greyish-white, medium- to coarse-grained sands were supplied by North German rivers (Harderwijk Formation, no. 3 in Fig. 1). The lower part of this formation consists of medium-grained sands with gravel. The Harderwijk Formation is overlain by coarse-grained river sands and gravels of the Enschede Formation (no. 4 in Fig. 1), likewise of northeastern provenance. In the basal part of this formation of Menapian Age a thin clay bed that closes the underlying aquifer is often present. At the start of the

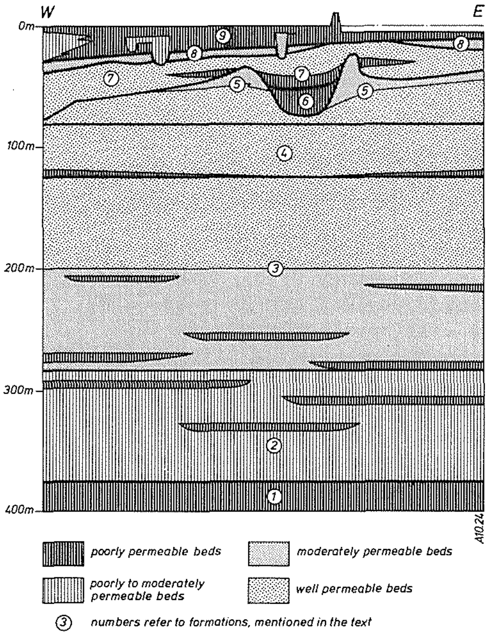

Fig. 1 Schematic cross-section through the study area

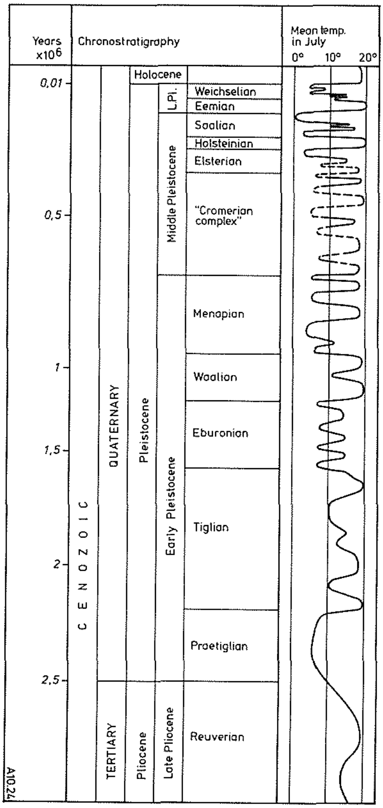

Fig. 2 Chronostratigraphy and paleoclimatic curve of the Quaternary (modified after Zagwijn, 1975)

Middle Pleistocene, during the Cromerian, the supply of clastic material from the northeast diminished in favour of a supply by the southeastern rivers (Rhine and Meuse), which resulted in the deposition of multicoloured river sands. In this area these fine-grained to coarse-grained sands are assigned to the Urk/Sterksel Formation (no. 5 in Fig. 1) of Cromerian, Elsterian, and Holsteinian Age. In the upper part of this succession fine-grained sands and silty clays may occur. These sands are part of a higher-lying aquifer.

During the Saalian, this part of The Netherlands was glaciated and was covered by large masses of inland ice. This glaciation resulted in the creation of glacial basins more than 100 metres deep, and ice-pushed ridges. Till was laid down by glaciers along the flanks and in the glacial basins, which after the retreat of the inland ice were filled with thick successions of varved clays (lake deposits). Together with medium- to coarse-grained fluvioglacial sands, these deposits belong to the Drente Formation (no. 6 in Fig. 1). During and after the withdrawal of the inland ice masses, which contributed considerably to the consolidation of the underlying unconsolidated deposits, the river Rhine found a westward path again. Multicoloured coarse-grained river sands (Kreftenheye Formation, of Saalian, Eemian, and Weichselian Age) with gravel of southeastern origin mixed with gravel of Scandinavian provenance were deposited in the study area.

In the next interglacial period, the Eemian, the sea-level rose and marine shell-bearing sands and clay beds were deposited (Eem Formation). Thick clay successions occur in places in the former glacial basins. At the end of the Eemian, the sea-level dropped and marine sedimentation was again replaced by fluvial deposition, now of coarse-grained gravelly sands of southern provenance (Kreftenheye Formation, together with the Eem Formation indicated as no. 7 in Fig. 1).

Fluvial sedimentation originating from the Rhine continued in this area well into the next and last glacial period, the Weichselian. During the Middle and Late Weichselian, fine-grained sands were transported by wind action and laid down as coversands (Twente Formation, no. 8 in Fig. 1). This resulted in a further smoothening of this area. The Kreftenheye Formation, the sandy parts of the Eem Formation, and the Twente Formation together constitute the uppermost aquifer in the study area.

3. Holocene

During the Holocene, sedimentation in the study area was strongly influenced by the post-glacial rise of the sea-level. Initially, this resulted in the development of a peat bed, Basal Peat or Lower Peat (De Mulder & Bosch, 1982), in the topographycally lower places. Because of its impermeability, this Basal Peat bed is of great hydrological significance. Due to the continuing rise of the sea-level the sea invaded this peat landscape and a brackish lagoon was formed in which clays were deposited. The coast line migrated further to the east and the brackish lagoon was replaced by a tidal-flat area composed of an irregular complex of mud flats and sandy flats dissected by numerous narrow channels and some broad main tidal channels. Deep scouring of the underlying clay beds, the Basal Peat, and the topmost Pleistocene sands, took place in the main channels. These tidal channels were filled up predominantly with sandy deposits, which permitted hydrologically unobstructed contact between the uppermost Pleistocene aquifer and the Holocene sands (see Fig. 3).

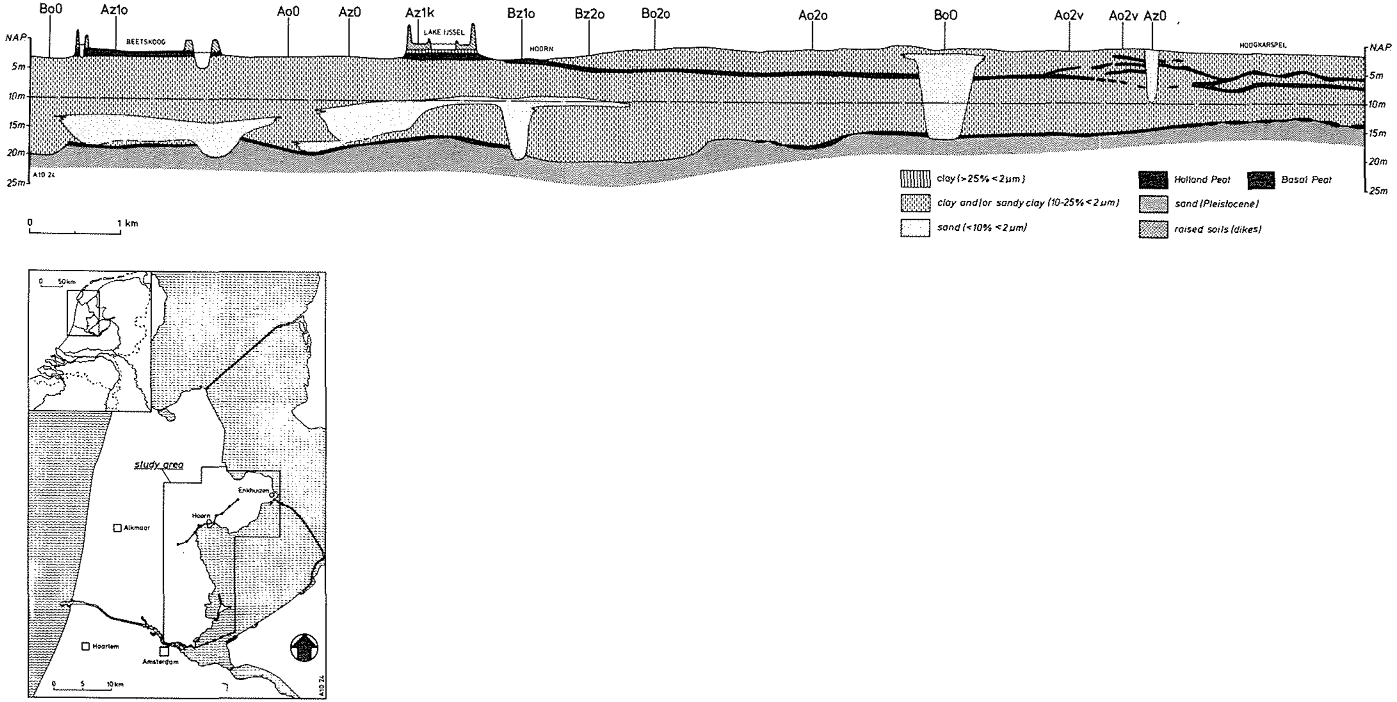

Fig. 3 Cross-section through Holocene deposits of eastern North-Holland, with some of the classification types (see Fig. 4) indicated

code	I	code	II	code	III	code	IV	Type	code	V	Type
A	*Basal Peat present*	z	*Holocene sand*	2	*peat<1m, with clay cover>1m*	v	*intercalated peat beds*	Az2v			
						o	*without intercalated peat beds*	Az2o			
				1	*peat>1m*	k	*thin clay cover>0,25m*	Az1k	s	*intercalated clay beds*	Az1ks
						o	*without thin clay cover*	Az1o	s	*intercalated clay beds*	Az1os
				0	*peat absent*			Az0			
		o	*Holocene sand absent*	2	*peat<1m, with clay cover>1m*	v	*intercalated peat beds*	Ao2v			
						o	*without intercalated peat beds*	Ao2o			
				1	*peat>1m*	k	*thin clay cover>0,25m*	Ao1k	s	*intercalated clay beds*	Ao1ks
						o	*without thin clay cover*	Ao1o	s	*intercalated clay beds*	Ao1os
				0	*peat absent*			Ao0			
B	*Basal Peat absent*	z	*Holocene sand*	2	*peat<1m, with clay cover>1m*	v	*intercalated peat beds*	Bz2v			
						o	*without intercalated peat beds*	Bz2o			
				1	*peat>1m*	k	*thin clay cover>0,25m*	Bz1k	s	*intercalated clay beds*	Bz1ks
						o	*without thin clay cover*	Bz1o	s	*intercalated clay beds*	Bz1os
				0	*peat absent*			Bz0			
		o	*Holocene sand absent*	2	*peat<1m, with clay cover>1m*	v	*intercalated peat beds*	Bo2v			
						o	*without intercalated peat beds*	Bo2o			
				1	*peat>1m*	k	*thin clay cover>0,25m*	Bo1k	s	*intercalated clay beds*	Bo1ks
						o	*without thin clay cover*	Bo1o	s	*intercalated clay beds*	Bo1os
				0	*peat absent*			Bo0			

A10.24

Fig. 4 Classification of the Holocene lithological succession

About 5000 years ago the coast line reached its easternmost position, which is situated only a few kilometres west of the study area. The predominant part of the clastic marine sedimentation then shifted from the southern towards the northern part of the study area, and this was followed by the development of extensive peat accumulations in the south (Holland Peat). In the northern part (Westfriesland), the marine clastic sedimentation, interrupted by the development of thin peat beds, continued until about 3000 years ago. Subsequently, peat growth took place in the entire area. From that time on, the former Zuiderzee area was part of a non-marine environment, in which there was a mosaic of peat islands and freshwater lakes with organic mud deposits.

During Medieval times the peat landscape was affected by the sea invading from the North, which initially resulted in the deposition on top of the peat of thin clay beds of limited distribution.
In spite of the construction of primitive dikes, peat areas of increasing size were lost by marine erosion, which was strongly triggered by reclamation and peat extraction in the Middle Ages.
This reclamation consisted of the introduction of a primitive drainage system, which lowered the ground water table and then led to settling and oxidation of the peat. Over the centuries this caused the disappearance of enormous volumes of the original peat, and at present the underlying clastic deposits are exposed at surface in West-Friesland (Borger, 1975). In the southern part of the study area, where the original peat accumulations were considerably thicker, lakes were created by the peat extraction and storm effects.
Reclamation of these lakes, where the clastic sediments still lay below the peat at the bottom, started in the sixteenth century but did not occur on a large scale until the seventeenth century. For more detailed information on the sedimentary history and lithostratigraphy of this area, reference is made to De Mulder & Bosch, 1982.

4. Classification of the lithological succession

For the purposes of this study, the composition of the Holocene beds, which are up to 20 metres thick, had to be expresses such that the succession of the geohydrologically and soil-mechanically relevant

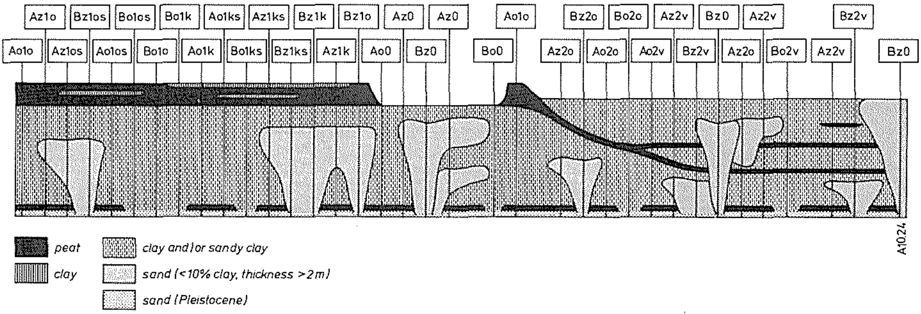

Fig. 5 Schematic cross-section with all classification types distinguished in the study area

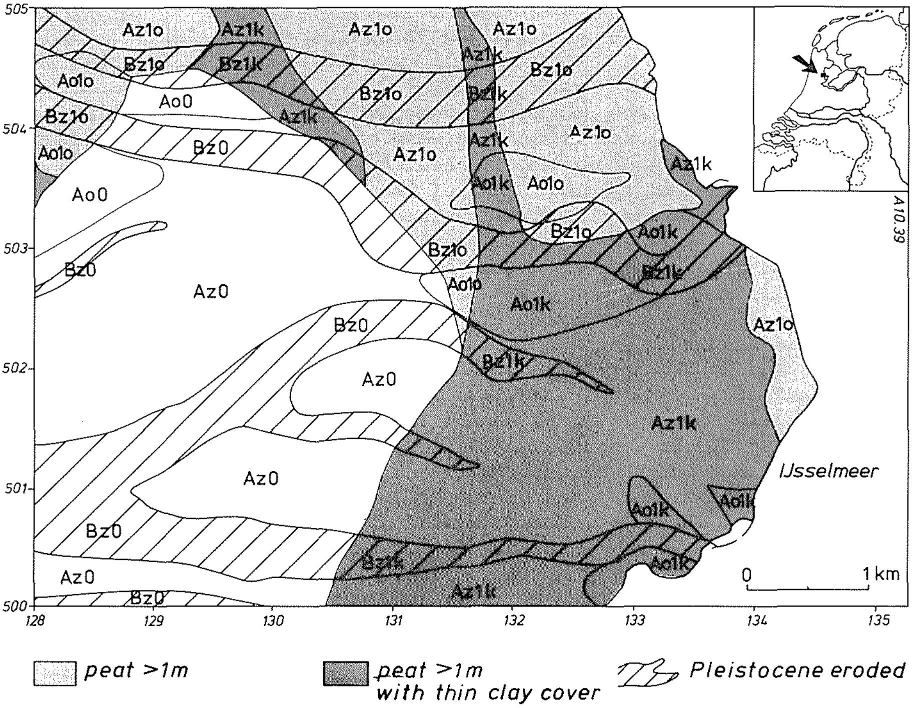

Fig. 6 Fragment of classification-type map

lithological units would be optimally indicated. This led to the use of the classification shown in Fig. 4. The composition of the Holocene cover beds is expressed in maximally five levels, which are related to geotechnically significant beds.

The highly impermeable Basal Peat bed is assigned to the first level of this classification, and the relatively permeable Holocene sand accumulations (thicker than 2 metres) to the second level. The Holland Peat and overlying clay beds occur on the third and fourth levels, at both of which several subdivisions are distinguished on the basis of relative thickness and occurrence of intercalated peat beds. Finally on the fifth level, there is the geotechnically important intercalated clay bed in a Holland Peat succession, which occurs in the southern part of the study area. Fig. 5 gives a schematic cross-section including all classification types distinguished in the study area. The distribution of the classification types is shown on maps on a scale of 1:25,000 (Westerhoff and De Mulder, 1981). Figure 6 shows a fragment of this map. Additional maps on the same scale show the depth of the top of the Pleistocene, the depth of the top of the Holocene sands, the depth of the base of the Holland Peat, and the thickness of the clay bed on top of the Holland Peat.

Acknowledgements

We wish to thank Dr. S. Jelgersma and Mr. J.B. Breeuwer for their comments; Mr. H. Bruinenberg for drafting the figures; Mrs. I. Seeger for reading the English text and Mr. I. Kraakman and Mrs. E.Y. Lamboo for typing the manuscript.

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GEOTECHNICAL CONSEQUENCES TO THE ENVIRONMENT, BY CONSTRUCTION OF THE POLDER MARKERWAARD

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Abstract

In the province of North-Holland, the reclamation of the polder Markerwaard will result in an increase of effective stress due to lowering of the water table and this will give rise to subsidence and settlement of buildings in the environment.

A probabilistic approach has been applied to calculate the expected settlements of the ground surface for an area of 500 km^2.

The time-dependent drop in piezometric level in the sand in both the polder Markerwaard and the province of North-Holland has been simulated using a suitable numerical model.

In addition calculations have been carried out to establish relationships between ground surface settlement and settlement of a foundation pile.

1. Introduction

Plans for the construction of the polder Markerwaard, in the Netherlands, take into account possible harmful effects in the province of North-Holland (Figure 1).

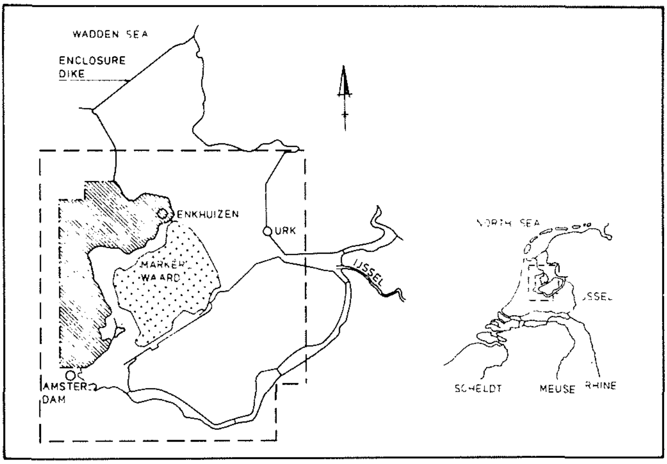

FIG. 1 The study area: Markerwaard and North-Holland

To forecast possible damage in that province, caused by induced changes in the groundwater regime, an integral study in relation to this problem has been set up (Claessen, 1984). The most important questions raised with respect to the geotechnical consequences were:

- What will be the magnitude and the rate of the settlements of the subsoil in North-Holland, due to a decrease of pore-water pressures in compressible layers?

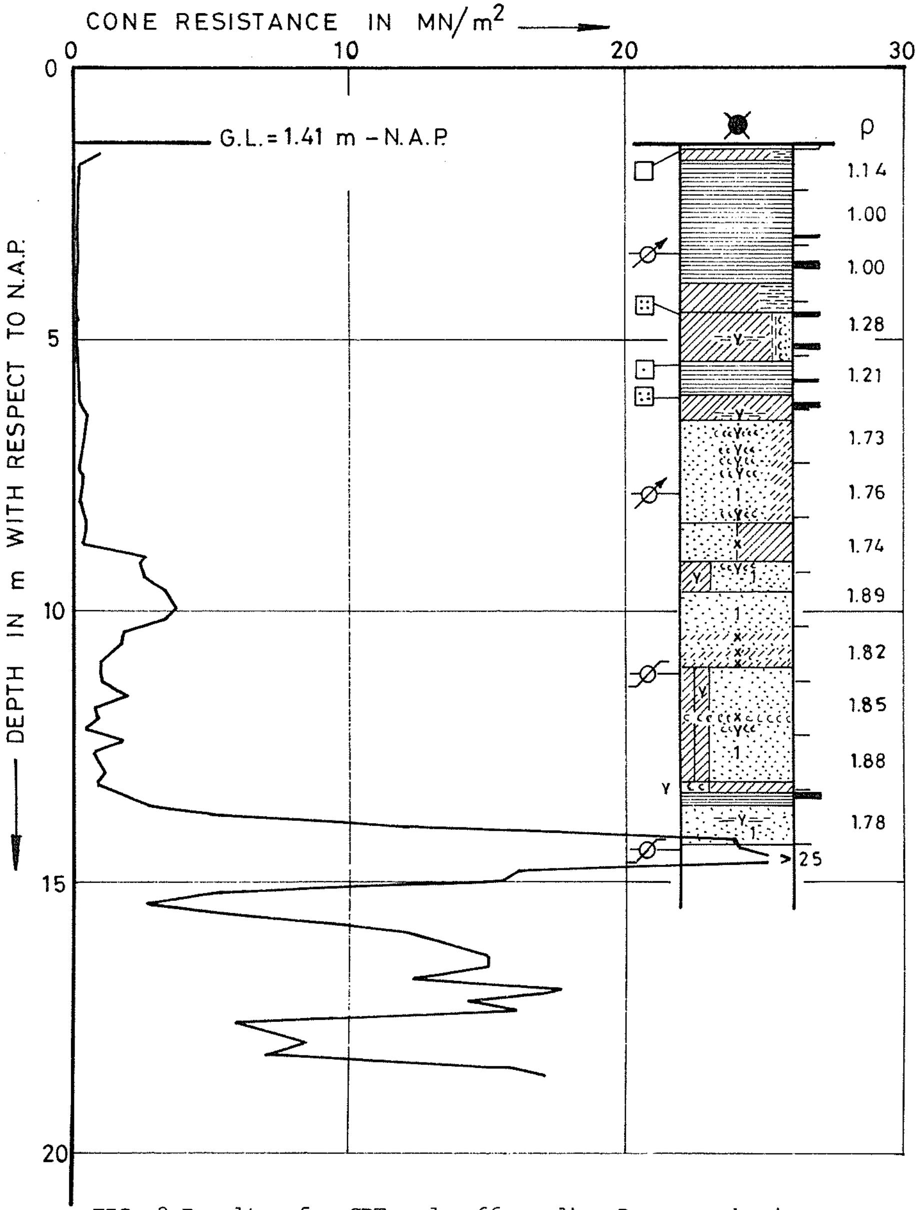

FIG. 2 Results of a CPT and a 66 mm dia. Begemann boring

- What relations are expected to exist between the settlement of the ground surface and the settlement of pile foundations in North-Holland?

The methods applied to answer these questions reflect the overall approach of the integral study: the study area of about 500 km^2 as a whole was considered.

No detailed studies of consequences for single buildings or small pieces of land were carried out.

2. Soil description

The subsoil in North-Holland consists of 10 to 20 m of Holocene compressible strata, overlying a sand deposit of Pleistocene age (Westerhoff and De Mulder, 1984).

All sediments of the Holocene in the study area belong to the Westland Formation. Apart from the layers found in the tidal gullies, the normal sequence of soil layers consists of the following main lithological units:

a) Lower peat: a thin layer of compact peat,
b) overlain by a clay deposit, followed by
c) a sand deposit, with intermediate clay layers and lenses
d) Holland peat: a rather thick layer of peat,
e) sometimes covered by a layer of clay.

In the tidal gullies the units a) and b) are missing. The gullies have as a rule, been filled by sandy deposits.

Figure 2 shows a representative soil profile, by means of the results of a Cone Penetration Test (CPT) and a 66 mm dia. Begemann boring.

3. Division of the study area in soil profiles

Settlement calculations are generally carried out for soil profiles in specific spots, where borings have been made. Results of laboratory tests on undisturbed soil samples taken from the borehole provide, in that case, the necessary geotechnical data.

Considering the magnitude of the area of about 500 km^2, this method would require an unacceptable amount of borings and laboratory tests. Therefore a probabilistic approach was followed using a model where areas instead of spots were considered.

In a first stage the whole study area has been divided in sub-areas with a uniform, geological lithostratigraphy. This division, based on available data consisting of results of borings and CPT's, is described by Westerhoff and De Mulder (1984).

In a second stage, the division in sub-areas with specific soil profiles, has been refined based on geotechnical considerations (Van Bruchem, 1984). This refinement has led to a further subdivision based on differences in

- layer thickness
- unit weights of the soil layers
- pore water pressure.

Thus approximately 150 soil profiles were distinguished.

4. Determination of geotechnical parameters

The soil description indicates a uniform constitution of the Holocene deposits in the study area. Therefore it can be justified that similar geotechnical properties are attached to soil layers in different areas which belong to the same lithological unit.

The two most important geotechnical properties in settlement calculations are compressibility and vertical permeability. The permeability of the different soil layers is important to the development of the pore-water pressure profile.

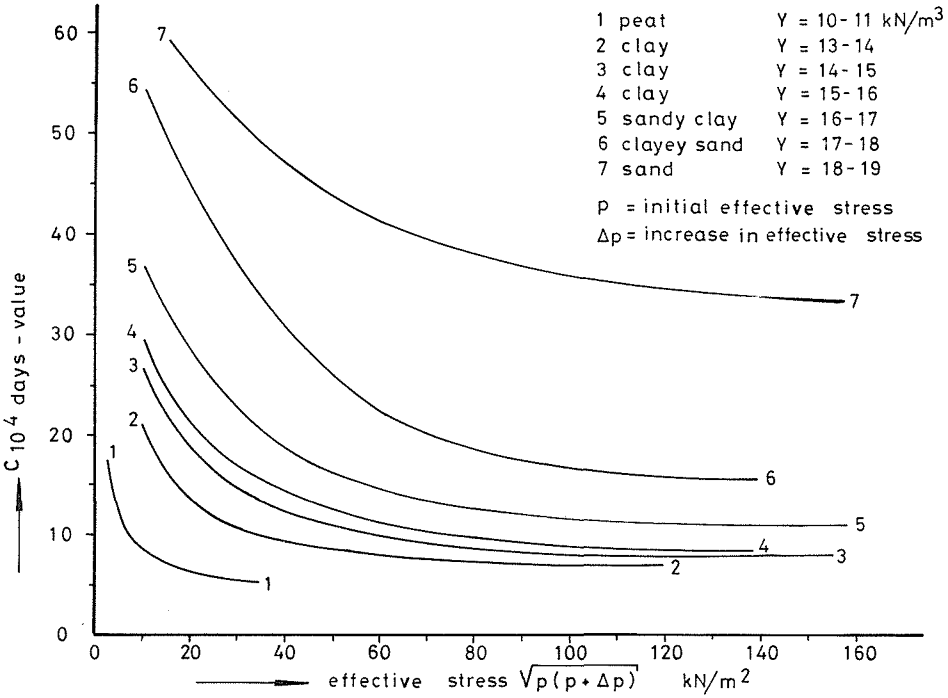

FIG. 3 Relation Compressibility Coefficient (C_{10} ^{4}-days) and Effective Stress

The compressibility determines the magnitude of the settlement due to an increase in effective stress in the compressible strata.

To determine the magnitude of these two parameters, results have been collected from all laboratory tests on soil samples obtained from borings carried out in the considered area. At the same time supplementary site and laboratory investigations have been carried out in those areas where insufficient data were available.

Figures 3 and 4 show the results of about 600 consolidation tests and 250 permeability tests after computer processing.

The presented curves show mean values for compressibility and permeability as a function of effective stress.

The curves have been used in the considered area to calculate the settlements of the 150 different soil profiles.

5. Soil settlement calculations

In the Netherlands soil settlements are usually calculated by the following formula (Koppejan, 1984):

$$s = H \left(\frac{1}{C_p} + \frac{1}{C_s} \log t\right) \ln \frac{p_o + \Delta p}{p_o}$$

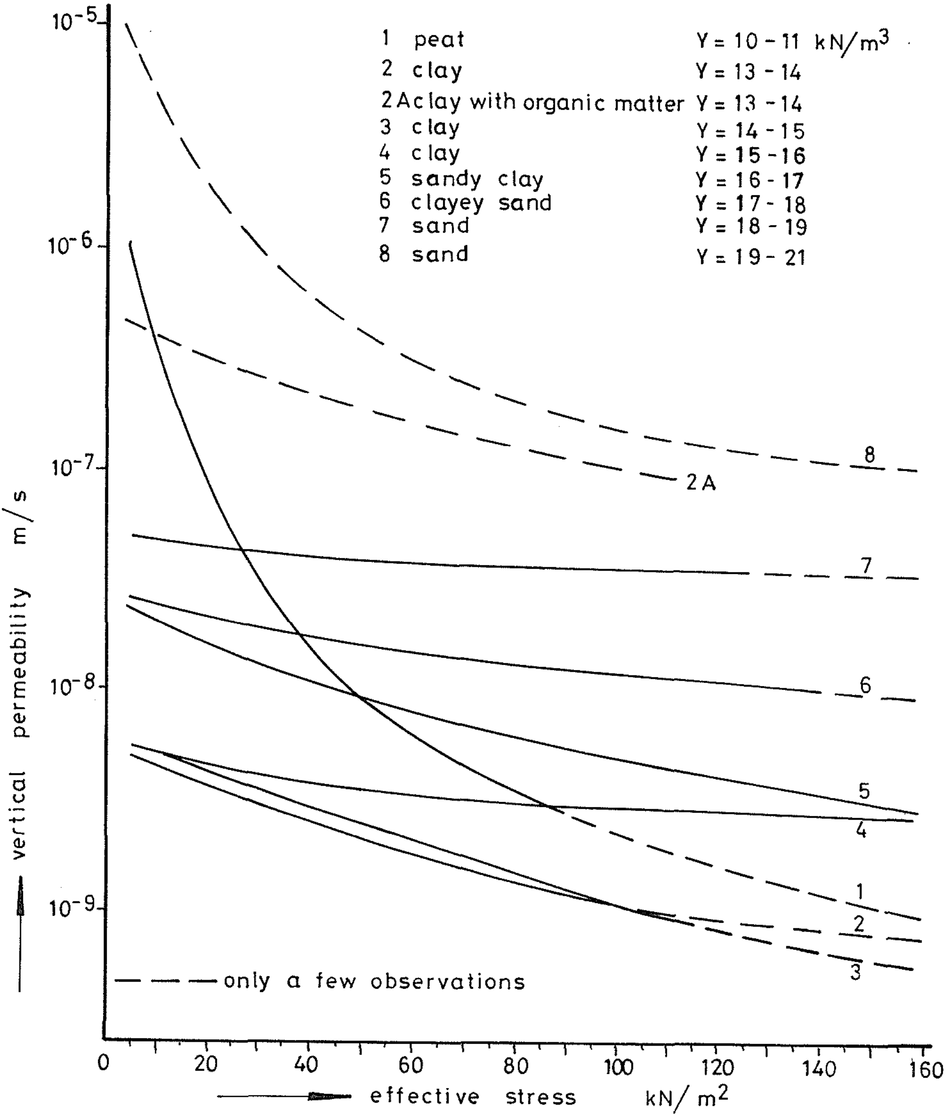

FIG. 4 Relation Vertical Permeability and Effective Stress

where:

s = settlement of a soil layer

H = initial thickness of a soil layer

C_p, C_s = compression coefficients <u>independent</u> of effective stress

t = time (usually in days)

p_o = initial vertical effective stress

Δp = increase in vertical effective stress.

The formula is often simplified to be able to calculate final settlements which are considered to take place in about 27 years (10.000 days) and reads then:

$$s_f = (H/C_{10^4 \text{days}}) \ln \frac{p_o + \Delta p}{p_o} \qquad (2)$$

where:

s_f = settlement of a soil layer in 27 years

$1/C_{10^4\text{days}} = 1/C_p + 4/C_s$

- Koppejan mentioned the possible non-validity of his formula in the range of low pressures. This possibility was taken into account in the present study by determining the $C_{10^4\text{days}}$-value from laboratory tests as a function of the vertical effective stress (Figure 3). The figure shows that the $C_{10^4\text{days}}$-values are indeed not constant at low pressures and therefore neither the $1/C_p$ and $1/C_s$-values.

The settlement of each soil profile due to a decrease in pore-water pressure can be calculated if both the present and future pore-water pressures are known:

. The present phreatic level is known from measurements. The calculations have been based on a present phreatic level of 0.20 m below polder water level (mean lowest phreatic level). The expected drop in the phreatic level was determined in another study (Van Bruchem, 1984).

. The pore-water pressure below the Holocene deposits is also known from measurements. The expected drop in the pore-water pressure was the subject of another study (Vos et al, 1984).

Knowing the geotechnical properties of each soil layer (Figures 3 and 4), the final settlement of each profile can now be calculated. The expected final settlements appear to have a maximum of 12 cm at some spots near the polder (Van Bruchem, 1984).

6. Reliability of the results

In order to obtain an idea of the reliability of the results of the settlement calculations, the following formula is considered:

$$S_c = \sum_{i=1}^{N} S_i = \sum_{i=1}^{N} \alpha_i \frac{H_i}{C_i} \ln \frac{p_i + \Delta p_i}{p_i} \qquad (3)$$

where:

S_c = calculated expected mean value of the settlement of a soil profile

S_i = calculated expected mean value of the settlement of a soil layer

α_i = reliability factor for a soil layer.

The factor α is added to the applied formula of Koppejan, and indicates the degree of correctness of the transposition of the soil data (borings and CPT's) into the different soil layers. The expected mean value of α equals in principle 1.0.

In analysing the available data the following coefficients of variation can be determined:

$V(H_i) = 0.20$

$V(p_i) = 0.20$

$V(\Delta p_i) = 0.25$

$V(C_i) = 0.04$ (due to the large amount of observations)

$V(\alpha_i) = 0.20$

Using equation 3 $\sigma(S_c)$, the standard deviation of S_c, is determined in a number of steps:

a) For each soil layer the expected mean settlement is calculated by:

$$S_i = \alpha_i \frac{H_i}{C_i} \ln \frac{p_i + \Delta p_i}{p_i} = \alpha_i \frac{H_i}{C_i} \frac{\Delta p_i}{p_i} \qquad (\Delta p_i << p_i) \tag{4}$$

This results in a value for the coefficient of variation of settlement for each soil layer:

$$V_i(S_i) = \sqrt{V^2(H_i) + V^2(p_i) + V^2(\Delta p_i) + V^2(C_i) + V^2(\alpha_i)} = 0.43 \tag{5}$$

and a value for the standard deviation of the settlement of a soil layer:

$$\sigma(S_i) = 0.43\ S_i \tag{6}$$

b) The expected mean value of the settlement of a soil profile is calculated by:

$$S_c = \sum_{i=1}^{N} S_i = S_1 + S_2 + \ldots + S_N, \tag{7}$$

resulting in a value for the standard deviation for the settlement of a soil profile:

$$\begin{aligned}\sigma(S_c) &= \sqrt{\sigma^2(S_1) + \sigma^2(S_2) + \ldots + \sigma^2(S_N)} \\ &= \sqrt{(V_1.S_1)^2 + (V_2.S_2)^2 + \ldots + (V_N.S_N)^2} \\ &= V_i \sqrt{S_1{}^2 + S_2{}^2 + \ldots + S_N{}^2} \text{ (provided } V_1 = V_2 = V_N) \\ &= 0.43 \sqrt{S_1{}^2 + S_2{}^2 + \ldots + S_N{}^2}\end{aligned} \tag{8}$$

c) The 150 different soil profiles were considered. The numbers of soil layers and the magnitude of the settlements of individual soil layers and of complete soil profiles, lead to minimum and maximum values for $\sqrt{S_1{}^2 + S_2{}^2 + \ldots + S_N{}^2}$:

$$0.32\ S_c < \sqrt{S_1{}^2 + S_2{}^2 + \ldots S_N{}^2} < 0.70\ S_c \tag{9}$$

Combining equation (8) and (9) leads to:

$$0.14\ S_c < \sigma(S_c) < 0.30\ S_c, \tag{10}$$

while the mean value of $\sigma(S_c)$ appeared to be $0.19\ S_c$.

The final results of the settlement calculations are presented as follows:

$$S_{RE} = S_c.(1 \pm 0.30) \tag{11}$$

where S_{RE} is the range of expectation for the settlements of a soil profile.

In conclusion this final result represents a reliability interval of 70 to 95%, with a mean value of 90%.

Thus, for an area with a determined soil profile the expected settlement may, with the above derived reliability, vary from 30% below to 30% above the calculated expected mean value.

7. Rate of drop of pore-water pressure

The rate of settlement in North-Holland depends not only on the soil properties in that area but also on the rate of drop of the pore-water pressure below the Holocene deposits. A number of calculations has been carried out to determine the drop of the pore-water pressure in the Pleistocene sand deposits as a function of time and place during and after the reclamation of the polder Markerwaard.

A simulation model has been developed to study this problem (Barends and Teunissen, 1984). The geo-hydrological situation is simplified into an axisymmetrical system.

The process of flow in the thick permeable Pleistocene sand deposits and the consolidation in the semi-pervious Holocene deposits are coupled. In the model the Markerwaard is represented by a circular polder with a radius of 11.5 km. The width of the boundary-lake between the new polder and the province of North-Holland varies from 1 to 6 km.

Figure 5 shows the results of a great number of calculations.
About 90% of the final drop in pore-water pressure in the Pleistocene deposits will take place in 5 to 7 years after the start of reclamation, if the drawdown in the polder will be in 0.5 year. If this drawdown will be in 3 instead of in 0.5 year, about 90% of the final drop in pore-water pressure in the Pleistocene deposits will take place in 7 to 9 years after the start of reclamation.

Figure 5 shows also the results of the calculations of the rate of settlement in the province of North-Holland. About 40 to 60% of the final settlement will take place in 3 years after the start of reclamation, depending on the speed of the drawdown.

8. Pile settlement calculations

The construction of the Markerwaard will cause a drop of the piezometric level in North-Holland in the sand stratum into which foundation piles have already been driven.
Therefore conditions will arise which are likely to induce additional negative friction forces on the piles due to the decrease of pore-water pressure in compressible layers around the pile shaft.

To assess the consequences of these additional forces, calculations were carried out for a number of single piles which were considered to be representative for the study area.

From calculations it was found that the expected settlement of the compressible soil layers decreases more or less linearly with depth (Figure 6).

The resulting negative skin friction can now be calculated for a pile with insufficient bearing capacity, if the pile is supposed to be incompressible (Heijnen, 1972).

Calculations of the resulting negative skin friction have been carried out where a relative movement between pile and soil was necessary to mobilize the full friction.

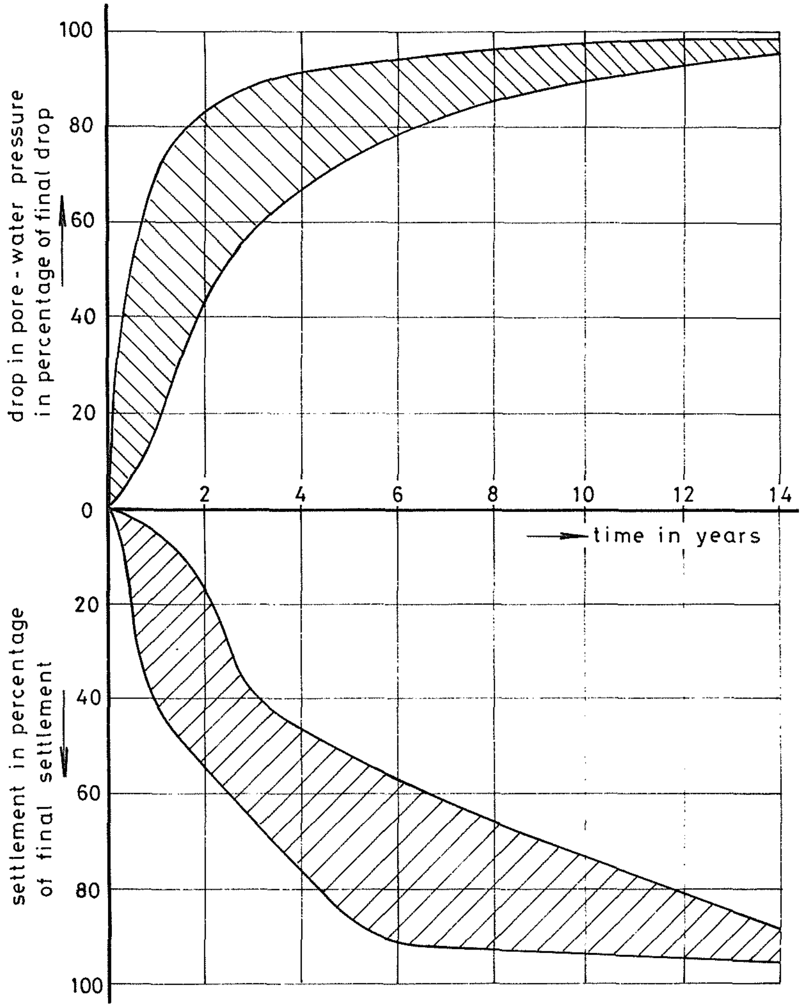

FIG. 5 Rate of the Drop in Pore-Water Pressure in the Pleistocene Deposits and the Rate of Settlement of the Holocene Deposits.

However, in the following simple model no relative movement was assumed to be needed even to mobilize this full friction.

Figure 6 leads to:

$$Q_{nr} = (1 - \frac{2\ S_p}{S_{gs}})Q_{nm}, \qquad (12)$$

where:

S_{gs} = settlement of the ground surface

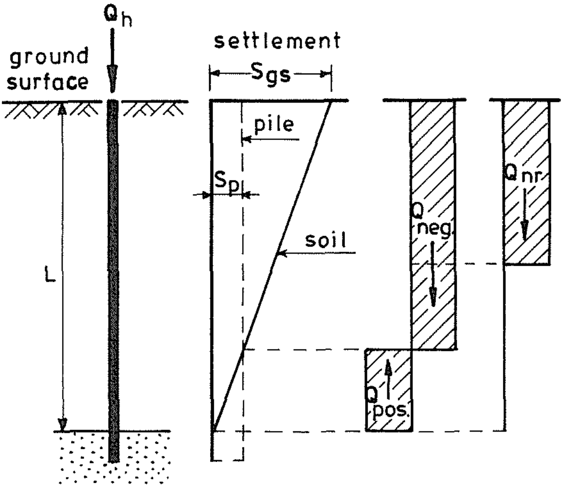

FIG. 6 Development of the Resulting Negative Skin Friction for a Settling Pile

S_p = settlement of the pile

Q_{nr} = resulting negative skin friction

Q_{nm} = maximum negative skin friction (that can act on a rigidly embedded pile)

Q_h = load on the pile head.

When the soil settles, the new equilibrium for a pile with insufficient bearing capacity can be found by considering the load-settlement behaviour of the pile. In this example a timber pile is considered. The ultimate bearing capacity of the sand stratum at the base level of the pile can be determined with the results of a CPT (Van Mierlo and Koppejan, 1952 and Van der Veen and Boersma, 1957).

When establishing the settlement of the pile at the state of equilibrium, the load on the pile that causes continuing settlement has to be assessed. Generally this situation occurs at 70 to 80% of the ultimate bearing capacity (Plantema, 1948).

In figure 7 the expected load-settlement behaviour of the timber pile is reproduced. On the horizontal axis the load on the pile-head is drawn (Q_h = 80 kN = constant), and added to this is the (theoretical) maximum negative skin friction on the pile (Q_{nm} = 100 kN).

The latter can be determined from CPT-results (Begemann, 1969). As can be seen in figure 6, the resulting negative skin friction = 0 when the settlement of the pile is equal to half the settlement of the ground surface.
In figure 7 the settlements of the ground surface (equal to twice the settlements of the pile) are drawn at Q_h = 80 kN (Q_{nr} = 0). Load equilibrium of the pile exists at the intersection point of the load-settlement curve and the line which represents the resulting negative skin friction.

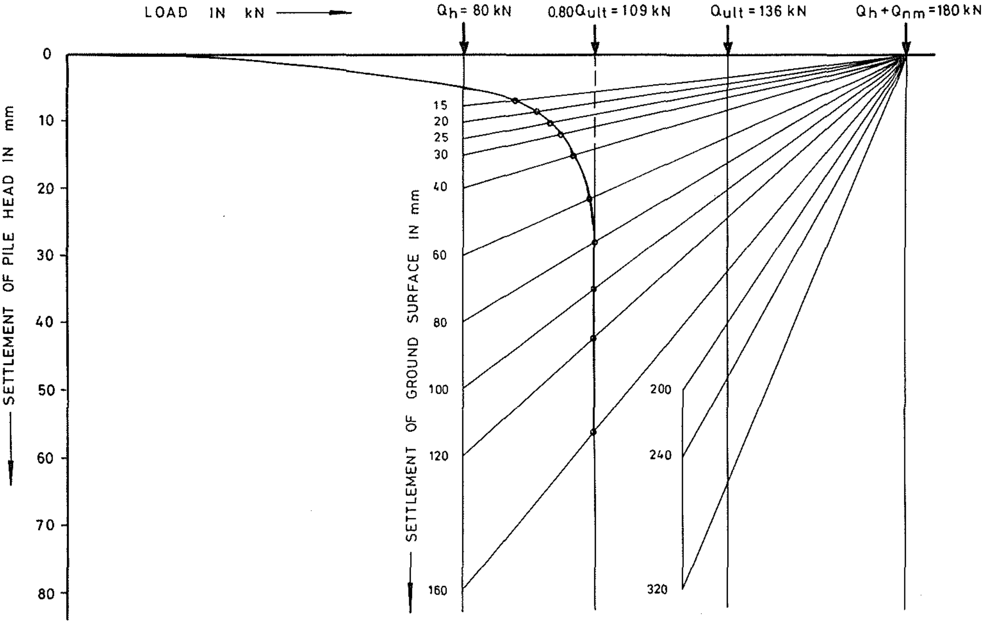

FIG. 7 Load-settlement Behaviour of a Timber Pile

By considering several assumed ground surface settlements, resulting in a number of intersection points, a relation can be established between the settlement of the ground surface and the settlement of the pile (figure 8).

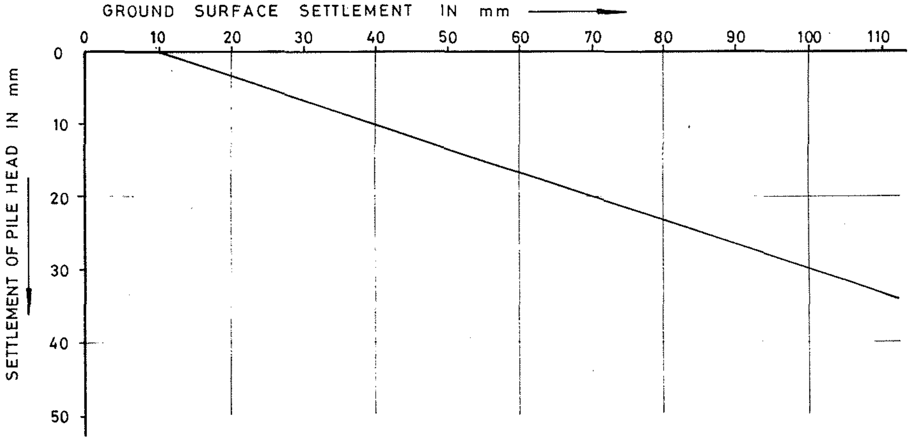

FIG. 8 Relation Between Settlement of the Ground Surface and the Settlement of the Pile

9. Results of pile settlement calculations

The results indicate that two different cases can occur:

I. $Q_h + Q_{nm} < 0.8\ (Q_p + Q_s)$, (13)

where:

Q_h = load on the pile head

Q_{nm} = maximum negative skin friction

Q_p = pile point resistance

Q_s = pile shaft resistance in the sand stratum.

The sand stratum will develop sufficient bearing capacity, without much settlement of the pile.

II. $Q_h + Q_{nm} > 0.8\ (Q_p + Q_s)$ (14)

The bearing capacity of the sand stratum is insufficient. Only when the pile settles to a certain extent, equilibrium in vertical direction is possible.

In the study area concrete piles will normally react as described for case I. However, timber piles will often react as described for case II.

According to the calculations the settlement of a timber pile may vary between 20 to 60% of the settlement of the ground surface.

10. Conclusions

Ground surface settlements in North-Holland have been calculated for a large area of about 500 km^2, based on probabilistic modelling.

The expected mean value of the average settlement of 150 considered soil profiles in North-Holland varies from 12 cm near the polder to 2-5 cm at 3-5 km more inland.

About 90% of the final drop in piezometric level will be achieved within 5 to 7 years from the start of reclamation. In that period 60% to 90% of the final settlement will take place.

Calculations have been carried out to assess the expected pile settlements in the study area. These calculations, based on a simple model, indicate for timber piles possible settlements of 20 to 60% of the ground surface settlements.

The results of the settlements of both the ground surface and the pile were used to determine the amount of damage that can be caused in urban areas (Carree and Hulsbergen, 1984), and to evaluate the measures required to prevent the drop in piezometric levels (Vos et al, 1984).

Acknowledgements

This study forms part of a project study by five research institutes.

The authors express their gratitude to Rijkswaterstaat (Dutch Public Works Department) for permission to present the results of the study made on their behalf.

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GEOHYDROLOGICAL ASPECTS OF THE RECLAMATION OF THE MARKERWAARD POLDER, THE NETHERLANDS. GEOTECHNICAL INVESTIGATIONS.

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Abstract

After reclamation of the Markerwaard polder, the changes in the groundwater regime will result in a settlement of compressible holocene layers (clay and peat layers).

Within the framework of the geotechnical investigations we have studied the spatial distribution of the settlements to be expected. To this end the study area was divide into subareas, which are, within certain limits, uniform as regards the profile structure of the holocene layer and the hydrological situation (map of geotechnical subtypes).

These calculations were carried out using the settlement theories of Terzaghi - Koppejan, modified to incorporate stress-dependent compression constants.

On the basis of the computational results we prepared 12 diagrams for the whole study area showing the relationship between the reduction in piezometric head in the first aquifer and the final settlement.

Using these diagrams, the map of geotechnical subtypes, and the piezometric head reduction known from the geohydrological investigations, a map was prepared on which the expected land subsidence (settlements) are indicated.

1) General

Hydrologic model investigations have shown that reclamation of the Markerwaard polder in the Netherlands will cause changes in the groundwater regime on the mainland in a 5-10 km wide strip along the eastern coast of North-Holland Province. These changes comprise decline of the piezometric head of the deeper groundwater and consequent minor declinations of the phreatic surface, especially in summer (Vos et al 1984).

As part of the investigations in North-Holland into the geohydrologic aspects of the reclamation of the Markerwaard polder, a study has been made of the behaviour of the subsoil in relation to the changes in the groundwater regime. This study, designated as 'geotechnical investigations', was done in close co-operation between the Geological Service of the Netherlands (RGD), the Delft Soil Mechanics Laboratory (LGM) and Heidemij Consultancy Division.

2) Problem definition

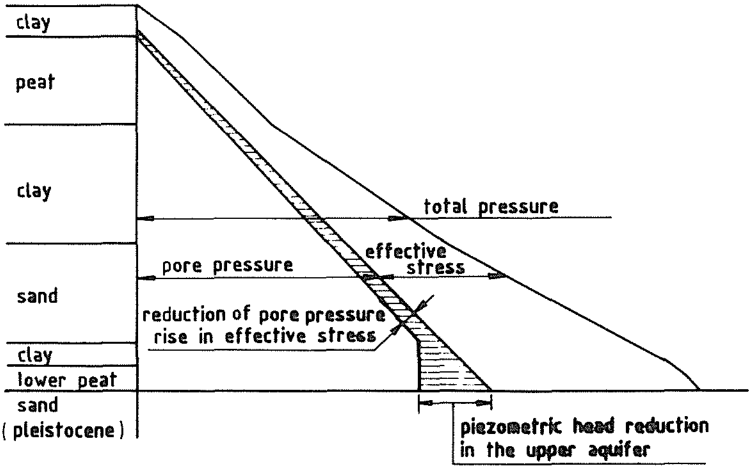
Under the influence of the decrease in piezometric head of the groundwater in the upper pleistocene aquifer and the decline of the phreatic surface, the pore pressure in the holocene layer will drop. As a result, the effective stress in this layer will increase, as is shown schematically in figure 1.

This increase in the effective stress in the holocene layer will result in settlement of compressible soil layers (clay and peat layers), so that

Fig. 1

Principal drawing

reduction in the pore pressure caused by a piezometric head reduction in the upper aquifer



subsidence of the ground surface will occur. The objective of the geotechnical investigations can thus be summarized as follows:

1. determining the magnitude of the settlements and ground surface subsidence to be expected in the study area;
2. determining the rate at which these settlements will occur.

3) Set-up of the investigations

The magnitude of the settlement occurring through the changes in the groundwater regime is dependent on:

- the drop in piezometric head in the upper aquifer;
- the declanation of the phreatic surface;
- the present hydrological conditions;
- the geotechnical properties of the soil layers in the holocene layer (e.g. permeability and compressibility)

The rate at which the settlement will occur depends on:

- the rate at which the piezometric head in the upper aquifer decreases and the rate at which the phreatic surface subsides.
- the structure of the holocene layer;
- the permeability and the compressibility of the various soil layers in the holocene layer.

In view of the above, it will be clear that soil investigations can give adequate information on the structure of the holocene layer, the hydrological conditions, and the geotechnical properties of the various soil layers in one single spot, but that there will be substantial variations within a large study area. For this reason the problem has been approached area-wise; the study area was divided into subareas which are, within certain limits, uniform as regards:

- the structure of the holocene layer;
- the hydrological conditions;
- the geotechnical properties of the various soil layers.

The Geological Service then prepared a basic map on which the study area was divided according to the structure of the holocene layer (Westerhof et al, 1984). This map was mainly based on information on file with the Geological Service, the Soil Mechanics Laboratory, and Heidemij. In places for which insufficient data were available, the Soil Mechanics Laboratory carried out additional soil investigations. These investigations showed that 11 typical soil layers can be distinguished in the holocene layer; in each of these layers the relevant geotechnical properties are fairly uniform.

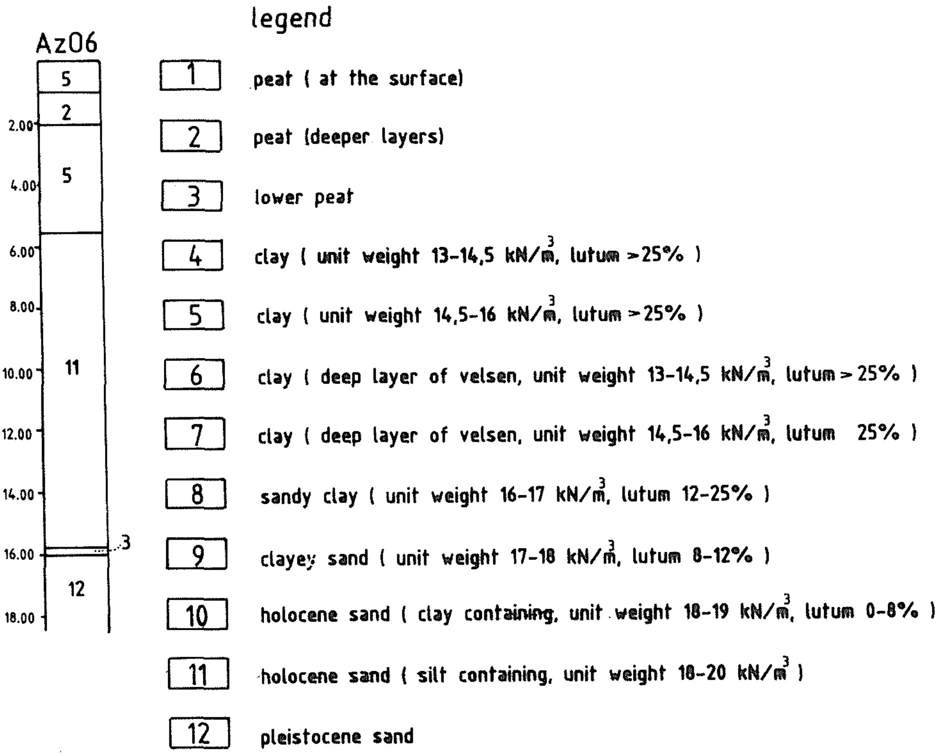
In addition, it was possible to establish clear relationships between the different geotechnical parameters (Hannink et al, 1984).

With the information thus obtained about the geotechnical properties of the soil layers and further information on the hydrological conditions in the study area, 160 profile types were distinguished within the holocene layer. On the basis of these profile types, Heidemij then refined the basic map to a map of geotechnical subtypes. Figure 2 shows one of the profile types; the figure clearly demonstrates the heterogeneity of the holocene layer in the study area.

4) Computations

For each profile type the relationship was determined between the reduction in piezometric head in the upper aquifer and the final settlement.

Fig. 2   An example of a profile type



The final settlement is defined as the settlement occurring 10.000 days after reclamation of the polder. To determine this relationship, settlement computations were made for piezometric head reductions of 0.25, 0.50, 0.75, 1.00 and 1.50 m in the upper pleistocene aquifer. The hydrological investigations had shown that such piezometric head reductions are possible in the study area.

The decline of the phreatic surface for each profile type was input as a percentage of the piezometric head reduction.

This percentage was determined by two model investigations for 9 profile types which can be regarded as representative for the study area:

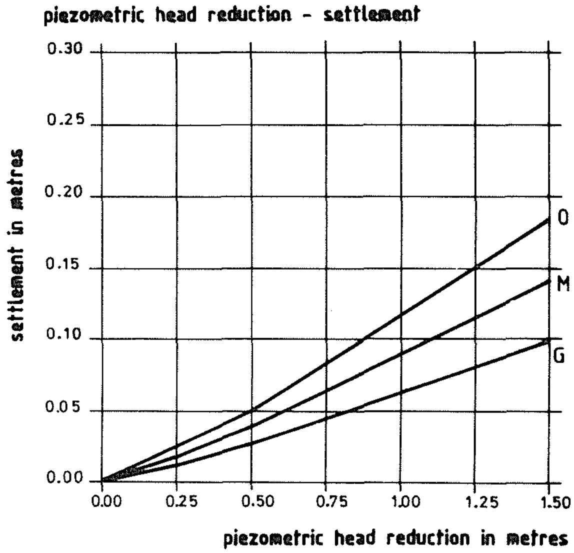
1. determination of the influence of the piezometric head reduction on the level of the phreatic surface in a stationary situation without rainfall or evaporation;
2. determination of the level of the phreatic surface during the growing season of a particular crop, as a function of the profile type and meteorological circumstances. In this non-stationary model a discharge relation is used that has been determined in the stationary model investigation (Heidemij consulting division; rapport nr. 662-83/1, 1983).

From the settlement computations it was found that the settlement behaviour of the study area can be represented in 12 diagrams, which give the relationship between the piezometric head reduction in the first aquifer and the final settlement.

The Soil Mechanics Laboratory carried out an analysis of the reliability of the soil parameters used; this analysis was used to define for each relationship diagram a band representing the 90 % reliability interval. Figure 3 shows such a diagram.

For the settlement calculations use was made of compressibility constants that were, unlike those in the usual theories, within certain limits dependent on the effective stress and the increment of the effective stress (Hannink et al, 1984).

Fig. 3 Relationdiagram



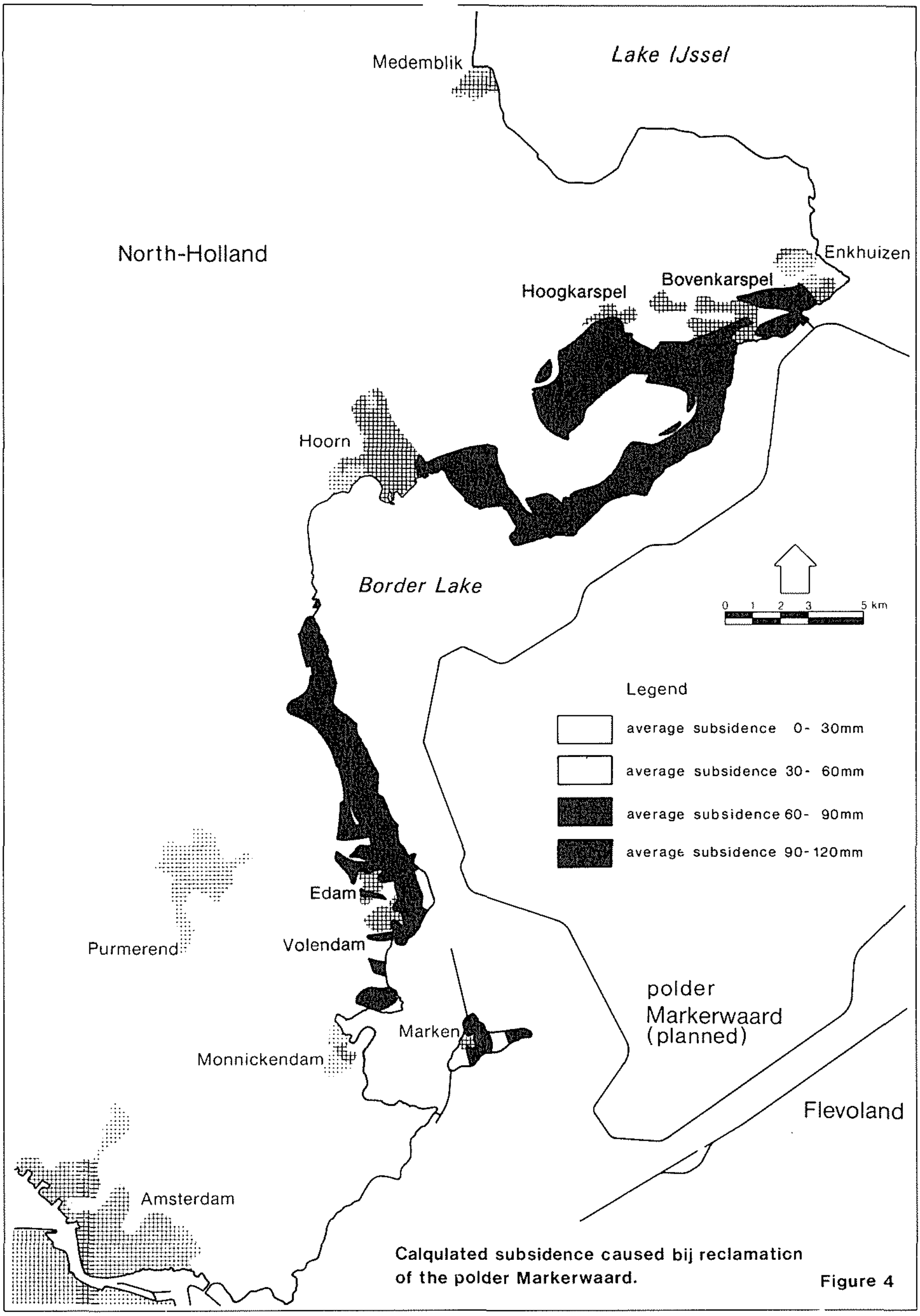


Fig. 4-Calculated subsidence caused by reclamation of the polder Markerwaard.

The rate of decrease of the piezometric head in the groundwater in the upper aquifer has a strong influence on the rate of the settlements. The Soil Mechanics Laboratory performed time-settlement calculations for the assumption that the piezometric head reduction will take place over a period of 5 years (Hannink et al, 1984).

5) Results

On the basis of the map of geotechnical subtypes, the piezometric head reductions determined in the geohydrologic investigations, and the diagrams presenting the relationship between piezometric head reduction and the final settlement, a map was produced on which the lowerings of the ground surface (settlements) to be expected are indicated (see figure 4). The map shows that the greatest settlements are to be expected immediately north of Volendam and in the vicinity of Wijdenes (0.09 to 0.12 m). Three to five km more inland settlements are expected of 0.03 to 0.06 m. In peat areas the lowerings of the ground surface may be somewhat larger than indicated on the map, as a result of shrinkage and oxidation of organic matter in peat profiles, due to lowering of the phreatic surface. This may cause an extra lowering of the ground surface of up to 0.02 m. It should be noted, however, that in the present situation the ground surface in peat areas is already subsiding due to oxidation of organic matter. Reclamation of the Markerwaard polder will accelerate this process by 1 to 2 mm per year for a number of years.

Since the rate of the piezometric head reduction in the upper aquifer has a much greater influence on the rate of settlement than the differences in the profile types, it was possible to represent the time-settlement relation for the whole study area by only 4 diagrams (Hannink et al, 1984). The computations show that within 5.5 to 8 years after the start of pumping the settlements will reach 90 % of their final value.

Acknowledgements

This study forms part of a project study by five research institutes.
The author wishes to thank Rijkswaterstaat (Dutch Public Works Department) for their permission to present the results of the study, and all members of the project-team for the pleasant co-operation.

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Cultuurtechnisch onderzoek.

"A METHOD FOR ESTIMATING BUILDING DAMAGE IN SUBSIDING AREAS"

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SUMMARY

Land subsidence, due to the withdrawal of water from compressible soils, may cause damage in urban areas. This situation can develop in the eastern part of Noord-Holland due to reclamation of the polder Markerwaard.
An estimate has been made about the expected damage for buildings and civil works.
An extensive survey was necessary in the area (500 km^2) with about 100,000 buildings to collect data for the number of buildings in each area, the types of building, construction dates, types of foundation etc.
Relationships have been found between the subsidence of the ground surface and the movement of the various types of foundation.
Standards for damage have been set up for the different types of building due to foundation movements.
Statistical analyses on the probability of damage, depending on the rate of settlement, for buildings and civil works have been used.
After accurately defining the patterns of damage, the reparation costs have been derived.
In this way it has been possible to develop, for each type of building, the relationship between the statistically expected damage costs (in Dutch guilders) per unit area (10,000 m^2) and the decline of the surface.

1. INTRODUCTION

A technical foundation research has been carried out within the framework of the integral research into the geohydrological aspects of the reclamation of the Markerwaard (Claessen, 1984). The purpose of the research was to estimate the expected damage in urban areas in the absence of counter measures.
Urban areas is taken to mean the built environment including infrastucture (roads, sewarage) in cities and villages.
The initial conditions for this estimate were given by the expected subsidence of the ground surface (Hannink and Talsema, 1984, Van Bruchem, 1984).
Our aim has been to produce a systematic and universally applicable calculation model with which the expected damage may be estimated.
Furthermore, standards for damage have been established for each characteristic type of building.
By using a complex electronic calculation model it is easy to calculate the expected damage for a change in the piezometric level eg in the case of subsidence of the ground surface in the presence of counter measures.
The expected damage is related to the total area in which possible damage could occur and is related to the existing characteristics of buildings and soil profiles.
In view of the character of this study it is not possible in this paper to go into details of the expectated damage.

2. MODEL FOR CALCULATING THE EXPECTED DAMAGE

The calculation of the expected damage has been carried out with help of electronic and drawing equipment.
A diagram of this model is shown below.

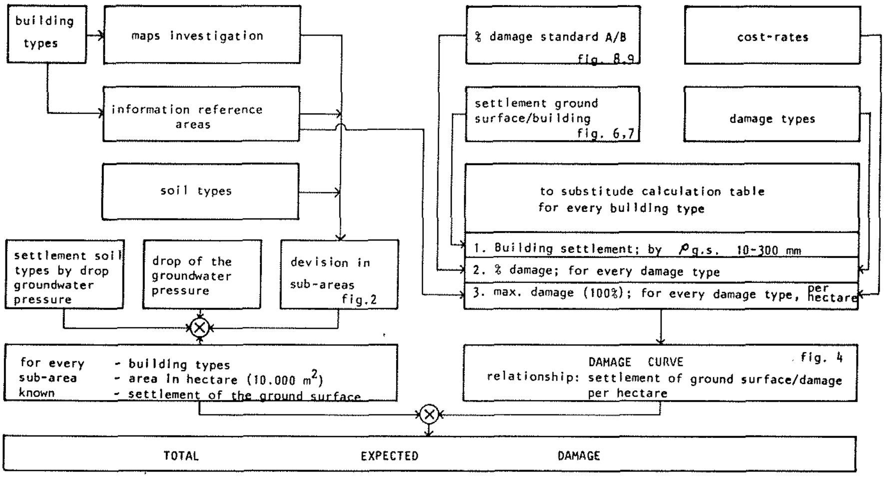
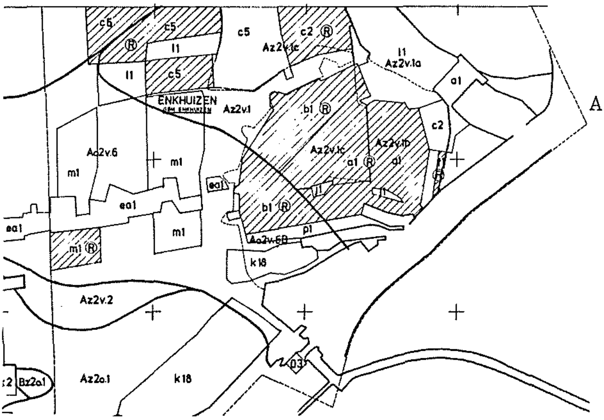


fig. 1. Diagram of the model for calculating the expected damage.

- <u>Investigation of urbanisation leading to a classification of the area.</u>

Based on file-data, topographical- and soil-use maps, the built area has been subdivided into areas of characteristic urbanisation. For the whole area 43 building types have been characterised in relation to: the type of building, age of building, the method of construction and the type of foundation. We have modelled areas exhibiting a high degree of homogenity such as estates. For such "reference areas" the relevant data relating to the buildings have been collected and reduced to, for example, the number of buildings per hectare (10,000 m^2)
The location, geographic boundaries and surface areas of the approximately 1200 distinct sub-areas have been digitalized with a drawing station. Furthermore, each sub-area has been allocated in associated building type and soil type.



R = reference area
K18 = building type
Az2v1 = soil type

fig. 2 Part of the drawing with area classification.

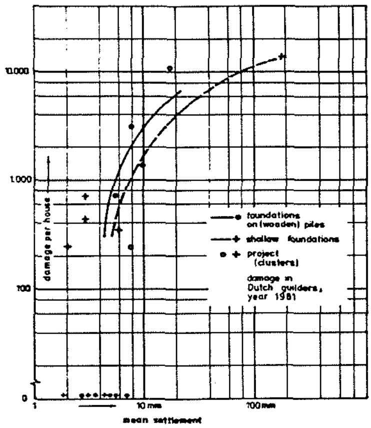


fig. 3. Relationship between building settlement and damage according to cases of damage.

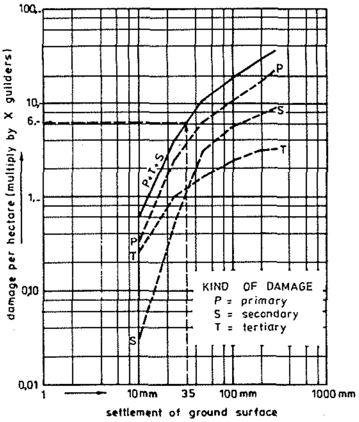


fig. 4. Example of a damage curve.

The different soil types are taken from the geological and geotechnical investigations (Westerhoff, et al 1984; V. Bruchem, 1984).

For each type of soil the relationship between the subsidence of the ground surface and the lowering of the height of soil water rise, established in the geotechnical investigation, has been incorporated (Hannink and Talsma, 1984, Van Bruchem 1984).

As variable, the levels with a certain lowering of the height of soil water rise have been digitalized by computer and recorded as x-y coordinate; they are derived from the calculated drop of the piezometric level in the first aquiferous stratum (Vos et al, 1984).

Intervals of 10 cm have been used.

The types of building and all their related characteristics, the area in hectares and the subsidence of the ground surface are obtained or can be calculated for each area from the above data.

- Standards for damage, leading to damage curves.

The damaging effects of a large number of drainage projects have been researched.

A total of about 5000 buildings, situated within the region of influence of the soil water drainage, have been considered. Of these about 12% had sustained damage.

The research has shown a relationship between the average building settlement and the average constructional damage appearing ; and between the total damage in an area and the total number of buildings, with or without damage. A distinction has been made between shallow foundations and wooden pile foundations (fig. 3).

Because of the large spread in the appearing damage it was not possible to define a statistical accuracy of the relationships found.

Standards for damage have been set up, for each type of building, based on literature information, the above mentioned cases of damage in the Netherlands, and a detailed estimate. The standards for damage result in a damage curve which, for each type of building, shows the expected relationship between subsidence of the ground surface and the damage per hectare (fig.4).

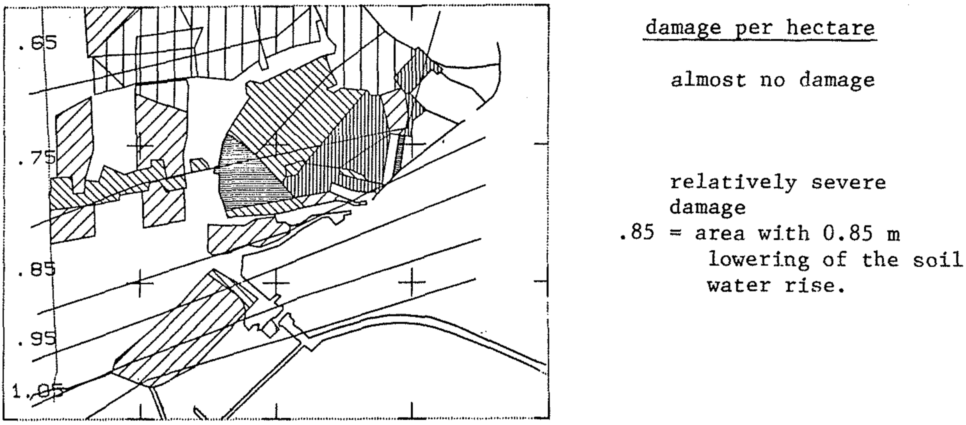


fig. 5. Part of a damage map.

A distinction has been made between primary damage (constructional damage such as crack forming, burst pipes, subsidence of roads), secondary damage (damage resulting from fall in value, advice and legal assistance, increasing road maintenance and industrial damage) and tertiary damage (costs for the government such as setting up and operating check points, administative costs of recording and rectifying the damage).
The way the standards for damage have been achieved is shown in par. 3.

- Calculation of expected damage.

For each area the expected damage can now be calculated. For instance for the area with building type C2 in fig. 2 (low rise buildings of brickwork, built after 1950, on wooden pile foundations) an area of 15 hectare and an expected subsidence of the ground surface of 35 mm is calculated. For this type of building the expected damage per hectare is 6x guilders (fig.4).
The total damage for this area is 15 . 6x = 90x guilders.
By summing all the areas the total estimated damage is determined.
To give a visual impression of the expected damage a damage map has been drawn up (fig. 5).
The hatching is a measure for the damage per hectare.

3. STANDARDS FOR DAMAGE

3.1 Introduction

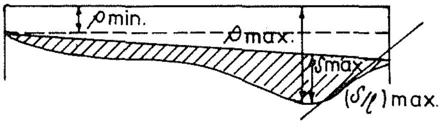
A calculation table is used for the determination of the damage curves (fig 4).
Based on the relationships between subsidence of the ground surface and the settlement of buildings, (as stated in par. 3.2), the settlement of the buildings has been determined for a ground surface subsidence of 10, 25, 50, 100, 200 and 300 mm for each type of building.
Different kinds of damage are distinguished for buildings and infrastructure.
Each kind of damage has been related to a type of building settlement and also associated with a particular standard for the expected damage (par. 3.3). Also for each kind of damage the cost rate has been determined, i.e. the costs of reparations per hectare (par. 3.3.).

The percentage of buildings for which damage is expected has been estimated for each kind of damage, for the various stages of ground surface subsidence (10 - 300 mm) and corresponding building settlement. (par. 3.3)
When for each stage of subsidence of the ground surface and for each kind of damage, the number of buildings where damage is expected is multiplied by the cost rate, a quantity of damage is obtained (par. 3.5).
Summing all the kinds of damage for a given subsidence of the ground surface, the expected damage is obtained. This results in a damage curve for a given type of building (fig. 4).
Damage standards were derived by making use of information in the literature (Grant et al, 1972; Burland et al, 1975 and 1978), the above mentioned practical cases in which damage occured as a result of drainage, and from office experience.

3.2 RELATIONSHIP BETWEEN SUBSIDENCE OF THE GROUND SURFACE AND SETTLEMENT OF THE BUILDING

In relation to the subsidence of the ground surface the following building movement have been distinguished:

ρmin = minimum or equal settlement
ρmax = maximum settlement of parts of the building
δmax = maximum differential settlement
(δ/ℓ) max = maximum angular distortion of parts of the building



For shallow foundation, wooden pile foundation and concrete pile foundation, the relationships between subsidence of the ground surface (ρg.s.) and the parameters ρmin, ρmax and δmax have been determined; (fig 6, 7).
Because of the rigidity of the superstructure and the load, distinction has been drawn between low-rise (1), middle high-rise (2) and high-rise (3).

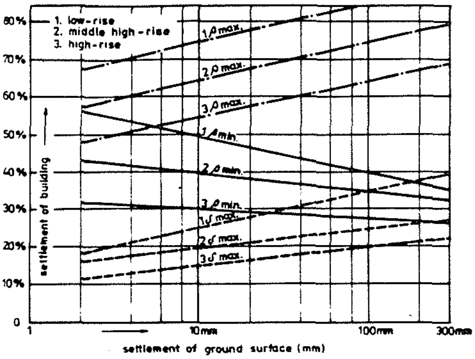


fig. 6. Relationship between subsidence of the ground surface - settlement of the buildings with shallow foundation.

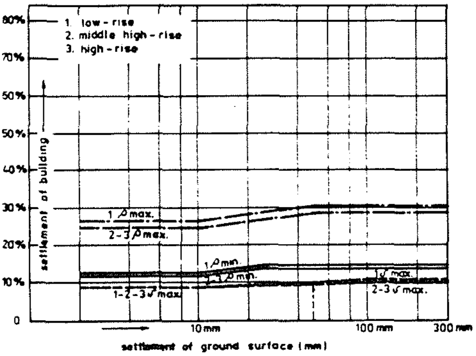


fig. 7 Relationship between subsidence of the ground surface - settlement of the buildings with wooden pile foundation.

For buildings with wooden pile foundation and concrete pile foundation, the conclusion of the geo-technical calculations in the geo-technical investigation was used (Hannink et al, 1984).

3.3 STANDARDS FOR PERCENTAGE EXPECTED DAMAGE

It was found that, for constructional damage, the maximum differential settlement δmax gives a better indication practically of the expected damage than the angular distortion (δ/l)max.

For low-rise, the discontinuities in the shape of wall-openings strongly determine the occurence of cracks. The relation between δmax and (δ/l) max is not explicit for low-rise because of the strongly varying value for l.

In fig. 8 the relationship between percentage expected damage and δmax is presented for the constructional damage to buildings.

Sometimes the damage is determined by the maximum subsidence ρmax (settlement of floors), the settlement difference between subsidence of the ground surface and building ρg.s. - ρmin (burst pipes) or the subsidence of the ground surface ρg.s. (roads); fig. 9.

Five damage classes have been distinguished from very light damage to very severe damage. The frequency of appearance of damage is dependent on the rate of settlement. The expected standards for damage shown relate to the average rate of settlement. Without counter measures the expected rate of settlement is "average": 18% of the final settlement (after 30 years) in 1 year, 42% in 2 years, 55% in 3 years.

At slower rate of settlement the expected damage is less; about half when approximately 30% of the final settlement occurs in the first 3 years.

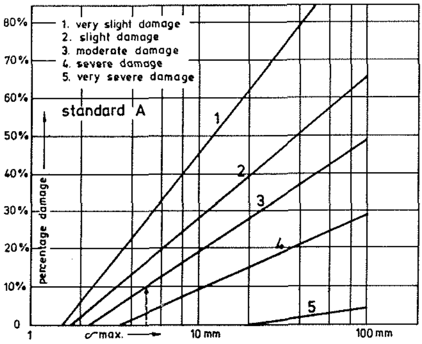


fig. 8. Percentage expected damage (Standard A).

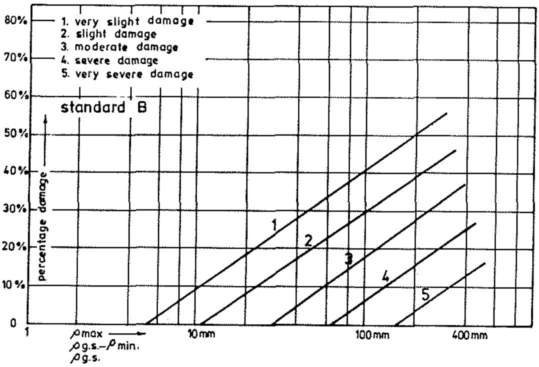


fig. 9 Percentage expected damage (Standard B).

3.4 COST RATES

In total 50 different categories of damage have been distinguished. For each categorie of damage the costs for repairs have been estimated on a per building basis; price per unit.

Based on counts in the reference areas (par. 2) the number of buildings (units) per hectare have been estimated. By multiplying the number of units by the price, the cost rates per hectare, for each category of damage, is obtained.

The cost rates are not the same for each type of building. Because more craft skill is required it is more expensive to repair a given damage in an old building than in a newer building; the price per unit is then higher. Because of the large number of types of buildings it is necessary, with respect to the cost aspect, to come to a standardization. For example factories are reduced to a certain number of standard buildings. The number of buildings is estimated from the building volume and wall surface of both factories and buildings.

3.5 EXAMPLE OF STANDARD FOR DAMAGE

- Type of building C2; subsidence of the ground surface 50 mm; average rate of settlement.
- *Kind of damage: small cracks in outside walls; costs of repair e.g.* ƒ 2000.= per building.
- Damage is estimated by δmax = 0.1 x 50 mm = 5 mm (fig. 7).
- Expected damage according to Standard A3 = 10% (fig. 8).
- Cost of repairs per hectare = 29 buildings x ƒ 2000 = ƒ 58,000.00
- Standard for damage = 10% of ƒ 58,000 = ƒ 5800 per hectare.

By estimating the standard for damage for each of the 50 kinds of damage, at 10, 25, 50, 100, 200 and 300 mm subsidence of the ground surface, and summing, one obtains the damage curve (fig. 4).

4. ESTIMATED DAMAGE WITHOUT COUNTER MEASURES.

Without counter measures, the expected subsidence of the ground surface in the urban area considered, is about 35 mm. In this case, the total damage will be about 800 million guilders (± US $ 250 million), with an error spread of 40% (price level 1981). The built area (total ± 80 km^2) has a value of approximately 20 billion guilders (± US $ 6.5 billion).

In the area are about 100,000 buildings.

Of the total damage about 85% will occur to buildings and about 10% to factories.

The total damage can be divided into 70% primary (constructional) damage, 15% secondary (not constructional)damage and 15% tertiary (administrative) damage.

With the same subsidence of the ground surface (average 35 mm), the constructional damage to a building of shallow foundation, will be twice as much as a building on wooden pile foundation, and 3 times as much as on concrete pile foundation; the expected average settlement of the buildings without counter measures will be ±24 mm, ±8 mm and ±4 mm respectively.

In addition to the calculation for the expected damage in the absence of counter measures, there has also been a calculation corresponding to partial counter measures; the drop of the piezometric level and the subsidence of the ground surface is less (Vos et al, 1984).

ACKNOWLEDGEMENTS

The study in this paper is part of a project study done by 5 research institutes. The project study was headed by the Ministry of Public Works, Management of Water-economy and Water-movement, District North.

The authors would like to thank the Ministry of Public Works for the opportunity to work on this study and for their guidance. They would also like to thank all the other members of the project for their kind cooperation.

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GEOHYDROLOGICAL COMPENSATORY MEASURES TO PREVENT LAND SUBSIDENCE AS A RESULT OF THE RECLAMATION OF THE MARKERWAARD POLDER IN THE NETHERLANDS.

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ABSTRACT

If the Markerwaard polder is reclaimed the water level in the polder will fall 5 to 6 metres over an area of 410 km^2. This will cause a drawdown of the piezometric levels in the Pleistocene aquifers underneath the eastern part of the province of North Holland. The spatial pattern of these drawdowns is calculated by a finite elements groundwater model.

Without countermeasures to compensate for the depletion of the piezometric level, settlement of the compressible Holocene clay and peat deposits will occur and resultant land subsidence may cause damage to buildings and infrastructure.

Drawdown of the piezometric levels can be entirely or partially countered by means of artificial recharge of water into the Pleistocene aquifers. There are two methods for this, viz. vertical recharge wells and infiltration grooves in the remaining western peripheral lake between North Holland and the Markerwaard polder. The amount of water necessary for the countermeasures is calculated with the same groundwater model.

1. INTRODUCTION

The study area is situated in the north-western part of the Netherlands (fig. 1). The projected polder will have an area of 410 km^2 and will be reclaimed in the south-western part of Lake IJssel.

The phreatic groundwater level in the new polder will be some 5 to 6 metres lower than the present water level in Lake IJssel. As a result, a groundwater flow towards the new polder will be induced. This will mean a considerable change in the groundwater regime underneath the new polder and its surrounding area. This change, particularly the drawdown of the piezometric level in the upper Pleistocene aquifer, could have a detrimental effect on agriculture and animal and plant life and may cause land subsidence, with resultant damage to buildings and infrastructure. This would be most likely to occur in the eastern part of the province of North Holland (the shaded area in fig. 1).

The province of North Holland can be characterized as an old polder area with a surface level varying from 1 to 4 metres below sea-level. The subsoil consists of Holocene clay and peat deposits 10 to 20 metres thick. (Westerhoff e.a. 1984).

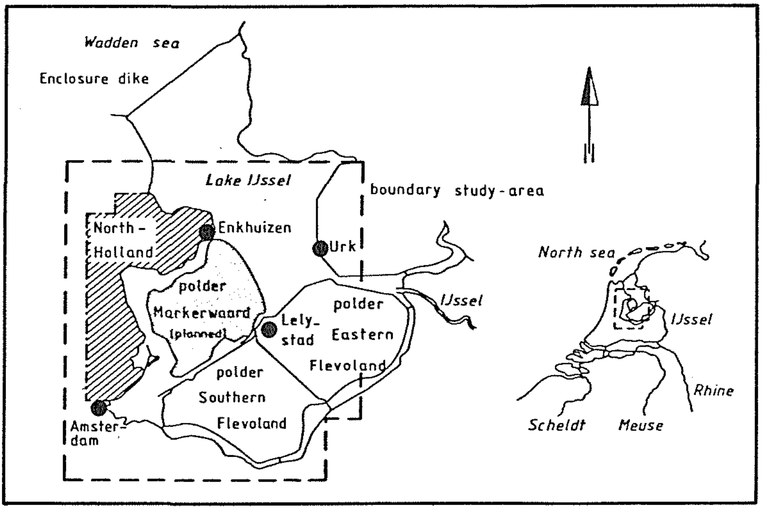


figure 1 Study area

The purpose of this investigation was firstly, to forecast the changes in the groundwater regime underneath the province of North Holland and secondly, to provide a feasibility study of countermeasures against the effects of such changes in the groundwater regime (Claessen, 1984).

2. GEOHYDROLOGICAL EFFECTS OF THE RECLAMATION OF THE MARKERWAARD POLDER

2.1. Introduction

Depending on the various geological and geohydrological conditions, reclamation of the polder will influence the existing groundwater regime. Because of the complexity of the geohydrological structure of the subsurface (in both a horizontal and vertical direction) together with strongly varying boundary conditions, the calculation of the groundwater flow had to be carried out with the help of a numerical model. Simple preliminary analytical calculations were used to determine the extent of the study area (= model area). (figure 1). The geohydrological features of the model were derived from the geological conditions of the study area (Westerhoff e.a., 1984). After calibration and verification this numerical model was able to forecast the changes in the groundwater regime.

2.2. Geohydrological schematization

For the geohydrological calculations it is necessary to schematize the subsurface into aquifers and aquitards, based on the geological conditions.

As shown in figure 2 the subsurface can be schematized into three aquifers covered and partially separated by aquitards. On the basis of the geohydrological framework a number of geohydrological maps

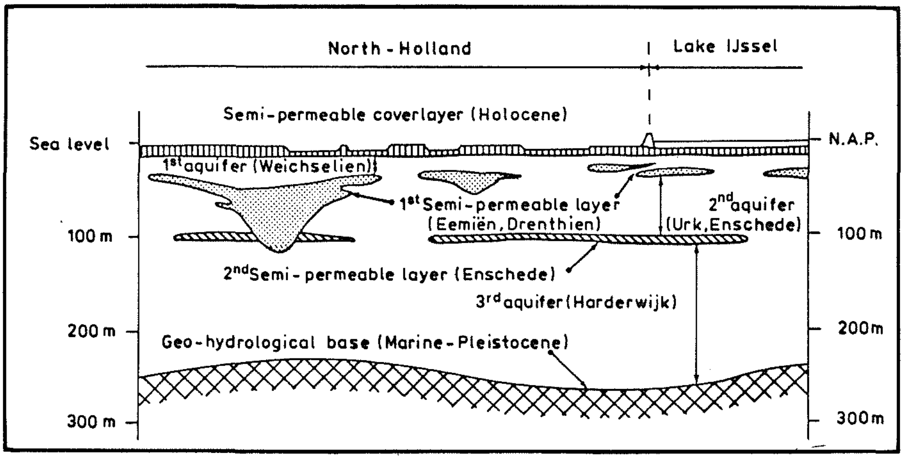


Figure 2 Geohydrological profile

have been prepared in which the preliminary values of the geohydrological parameters needed for the model computations, such as values of transmissivity (T) and vertical hydraulic resistance (c), are given. The parameter values firstly used are readjusted by means of model calibration.

2.3. Calculations with the finite element model

2.3.1. Description of the model

To forecast the changes in the groundwater regime after reclamation of the Markerwaard, the FIESTA-model was used. This is a numerical physical finite element model which solves steady groundwater flow problems in multi-layered systems (Public Works Department, 1978). The finite element discretization is based on the regional character of this study and the expected effects of the reclamation of the Markerwaard polder (figure 3).

The FIESTA-model calculates the piezometric levels and vertical groundwater flow for each nodal point of the three aquifers. In addition, water balances are calculated for the separate aquifers, the entire hydrological system and for those hydrological units that could be distinguished.

2.3.2. Calibration and verification of the model

For the calibration of the model (=parameter evaluation) the average piezometric levels over the period October/November 1978 were used (these being representative of a long term steady state situation), as well as infiltration and seepage values measured in the field.

The output of the model calculations, in terms of piezometric levels and vertical groundwater flows, should agree, within certain limits, with the measured data.

The geohydrological parameters in the model (T- and c-values) have been adjusted to an optimum value by a selective trial and error method. During the calibration process it became increasingly clear that the vertical hydraulic resistance of the Holocene

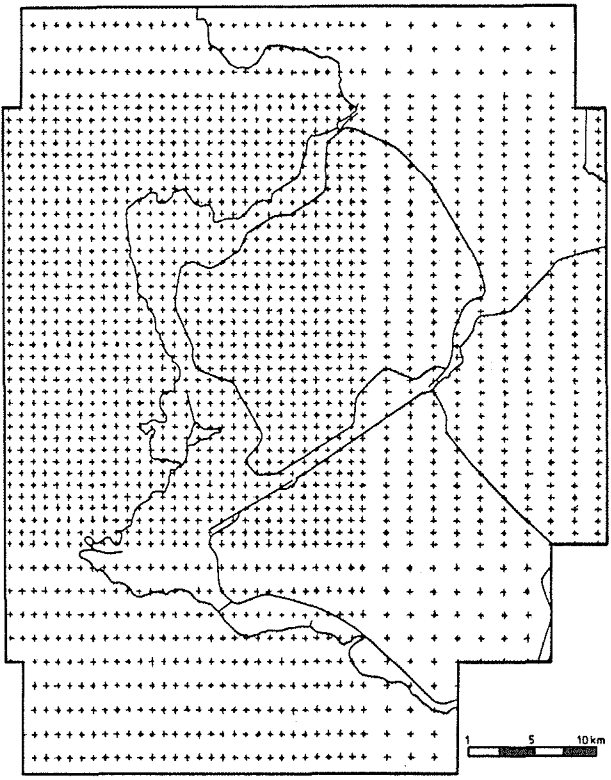


figure 3: Model area; part of grid spacing

aquitard is the most sensitive parameter for the groundwater flow of the area involved. Figure 4 shows the spatial distribution of this c-value as determined by the calibration of the model for the existing hydrological situation.

After calibration, the forecasting ability of the model was verified with data, different from those used with the calibration. In 1967 the Southern Flevoland polder (in the southern part of the model area) was reclaimed. This resulted in a drawdown of the piezometric levels in that particular region. The actual extent of the depletion was then compared with the predictions of the numerical model with fairly similar results.

2.3.3. Results of the calculations

To indicate the final situation (steady state) after the reclamation of the Markerwaard polder the input data of the model were adjusted as follows:

- in the new polder area the phreatic level drops 5 to 6 metres;
- in the same area the vertical hydraulic resistance of the Holocene aquitard decreases through ripening of the top soil and reclamation works. It was estimated, based on experience in other polders, that

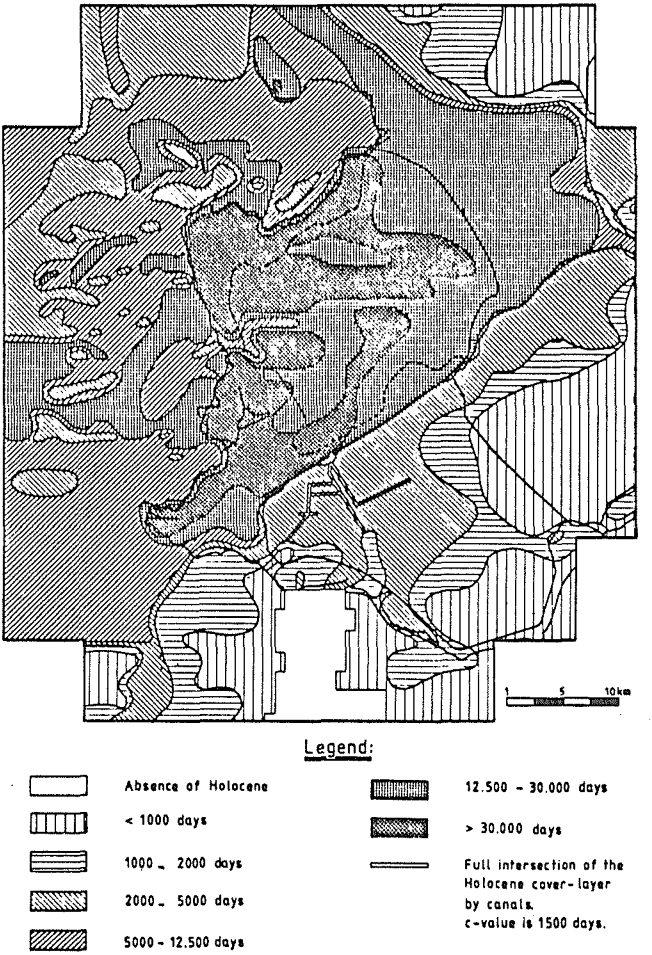


figure 4

Vertical hydraulic resistance (days) of the Holocene cover-layer.
(Situation before reclamation of the Markerwaard polder)

the c-values would decrease from about 27000 days to 4000 days.

After these adjustments, the new piezometric levels and vertical groundwater fluxes in the study area were calculated. Subtraction of these values from the corresponding values at present, results into the drawdowns of the piezometric levels in the three aquifers and the changes in infiltration and seepage rates.

The drawdowns in the upper Pleistocene aquifer (figure 5) is especially important, because these data are important for the geotechnical calculations necessary to forecast land subsidence and subsequent damage.

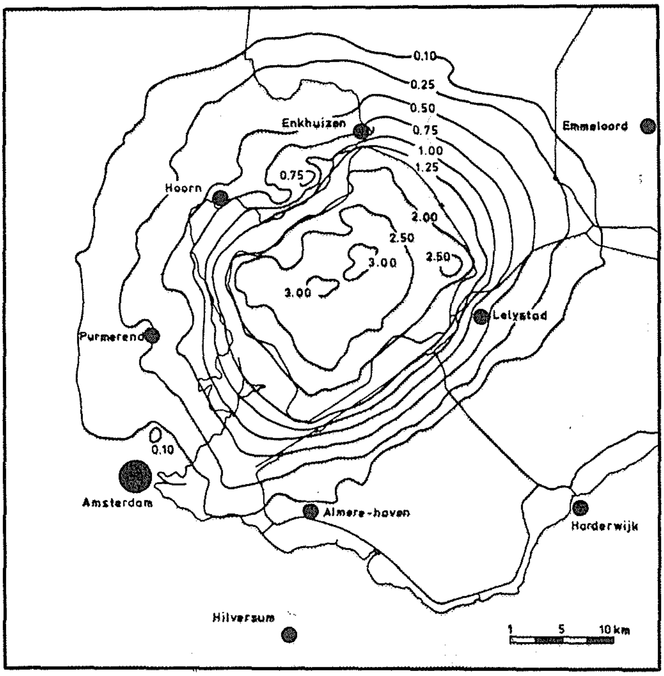


figure 5 : *Ultimate drawdown in upper aquifer.*

2.3.4. The speed of piezometric drawdown

Calculations have been made using an analytical non-steady groundwater flow model (Barends, 1984) to estimate the speed of the piezometric drawdown in relation to the speed of pumping dry the new polder.

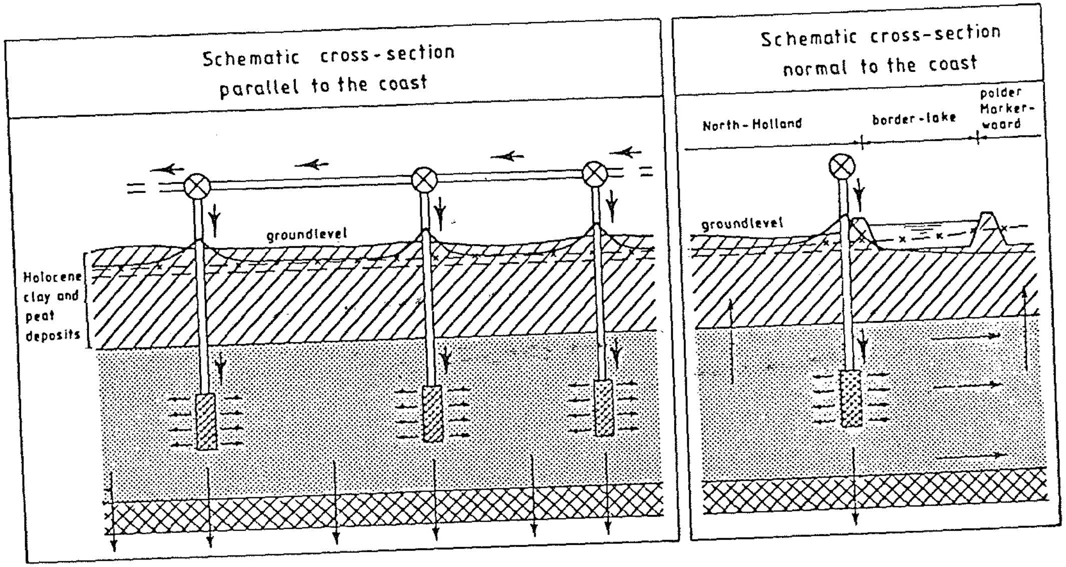
In this model the consolidation of the compressible Holocene deposits has been taken into account as a function of time and place. It appears from these calculations that under the Markerwaard and North Holland the piezometric level will recede at the same rate as ascertained under the north-western part of Southern Flevoland during and after the reclamation of this polder. In Southern Flevoland and its surroundings measurements showed that one year, respectively five to seven years after the start of pumping, more than 50%, respectively almost 100% of the final drop of the piezometric head was reached.

If the Markerwaard is pumped dry at the same rate as Southern Flevoland, it is to be expected that after 5 to 7 years about 90% of the final drop under and around the Markerwaard will be reached. (Hannink e.a., 1984)

3. COUNTERMEASURES TO PREVENT DRAWDOWN OF PIEZOMETRIC LEVELS

3.1. Introduction

Without countermeasures settlement of the compressible Holocene deposits will occur. Due to resulting land subsidence, buildings and infrastructure may be damaged (Hannink and Talsma, 1984; van Bruchem, 1984; Carree and Hulsbergen, 1984).



—x—x— piezometric head, situation before reclamation polder Markerwaard.

— — — — piezometric head, after reclamation, without counter measures.

———— piezometric head, after reclamation, with recharge wells.

figure 6

Sketch of application recharge wells.

3.2. Countermeasures studied

Drawdown of piezometric levels can be countered by means of artificial recharge of water into the Pleistocene aquifers. Various measures such as gas(air) injection, injection of chemicals, construction of slurry walls, sandpiles, infiltration galleries, recharge wells and infiltration grooves, have been briefly considered and analysed provisionally with regard to their technical, practical and economic feasibility.

Two solutions were found feasible for this specific problem and have been studied more in detail:

- injection of water by means of vertical recharge wells (fig. 6). These wells can be located along the east coast of North Holland (fig. 7) or clustered near those urban areas most vulnerable to damage (fig. 8).
  Depletion can be entirely or partially compensated. Water can be taken from the peripheral lake and after purification pumped into a transport system towards the recharge wells;
- increase of the infiltration capacity of the western peripheral lake by dredging infiltration grooves (10-15 metres deep) which would intersect the covering Holocene deposits at several places (fig. 9).

A third probable solution was derived from the results of the integrated study:

- slowing down the rate of drawdown by phasing out the reclamation works, or, by lowering the piezometric level gradually by phasing out the recharge well system over a period of about 30 years. Establishing a steady state in a period of 30 years will result in a very gradual settlement of the Holocene deposits, so damage to buildings may be greatly reduced.

3.3. Calculation methods

The amount of water necessary for complete or partial compensation by recharge wells and infiltration grooves was also calculated with the FIESTA-model. A model in which relations have been established between drawdown of the piezometric levels, land subsidense and damage to buildings and infrastructure (Carree e.a., 1984) calculated the residual damage for partial compensatory measures.

3.4. Results of the calculations

When using recharge wells to compensate fully for the drawdown of the piezometric levels in North Holland, a yearly amount of 40 million m^3 infiltration water is necessary, i.e. about 215 recharge wells. To compensate only for the drawdown in the regions most vulnerable to damage, a yearly amount of 20 million m^3 is required, or about 125 recharge wells.

For optimum compensation of the drawdown by means of infiltration grooves, the infiltration area of these grooves should be 8 km^2 or 24 km^2, in the case of a hypothetical infiltration resistance of 100 and 300 days respectively.

Fig. 10 gives the relations between the average drawdown per area of the piezometric level in the areas vulnerable to damage and the total quantity of injection water. Fig. 11 relates the calculated residual damage to buildings and infrastructure in North Holland to

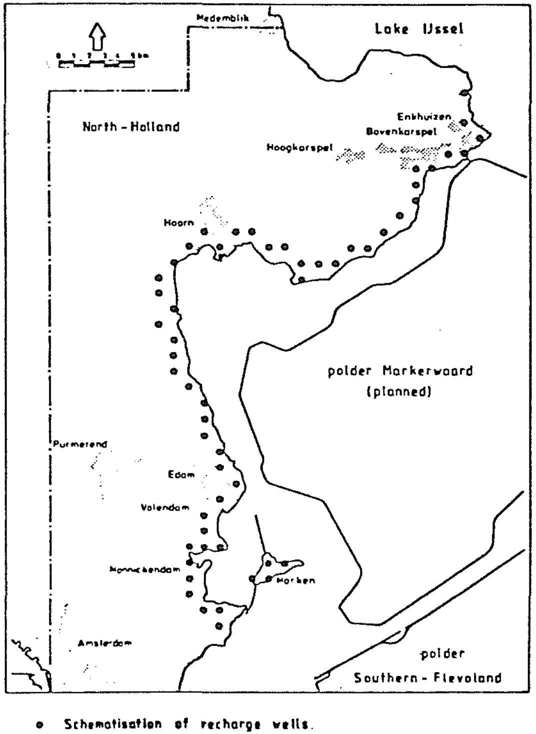


figure 7

Location of the recharge wells for infiltration in 3 aquifers to compensate fully the drawdown of the piezemetric level in North Holland,

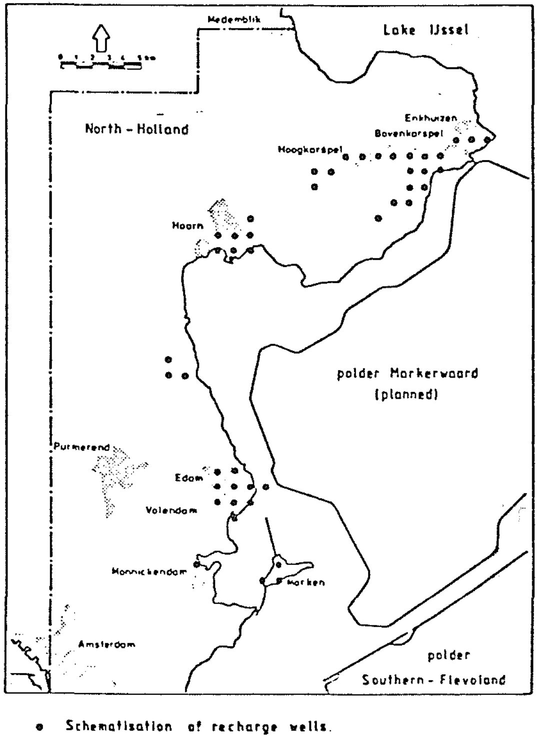


figure 8

Clustered locations of recharge wells for infiltration in the shallow aquifer to compensate fully or partially the areas vulnerable to damage.

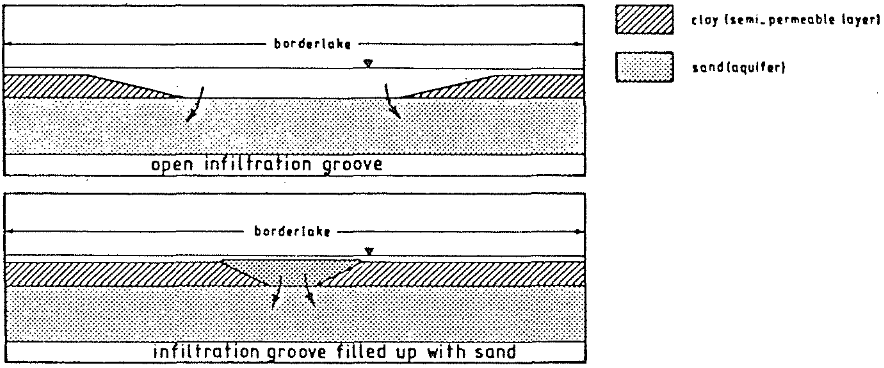
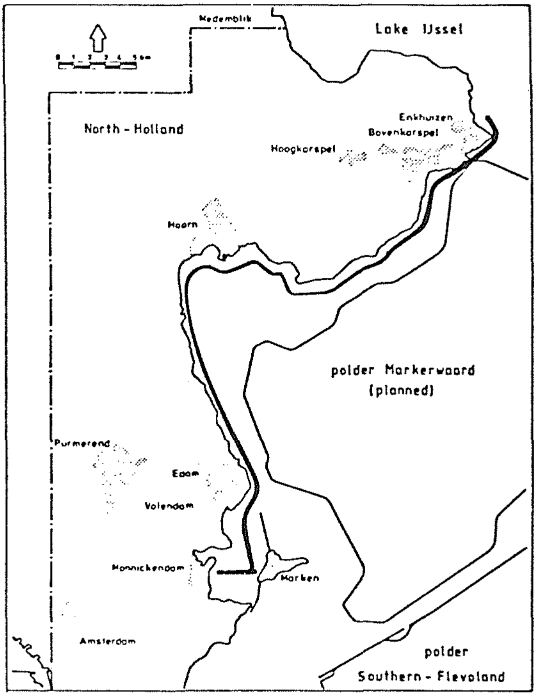


figure 9^a

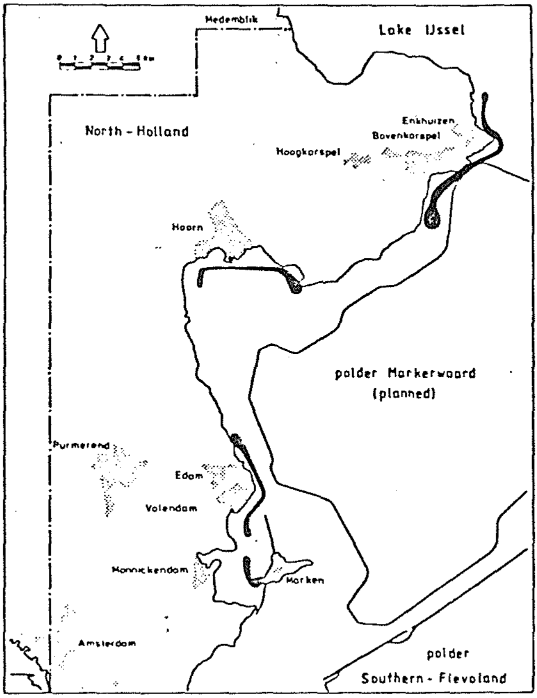
Schematic cross-section of infiltration grooves.



Schematisation of infiltration groove.

figure 9^b

Location of infiltration groove in the borderlake to compensate fully the drawdown of the piezometric level in North Holland.



Schematisation of infiltration areas.

figure 9^c

Locations of infiltration areas in the borderlake to compensate, fully or partially, the drawdown of the piezemetric level in the areas vulnerable to damage by land subsidence.

the total quantity of injection water, in both diagrams, both for recharge wells and infiltration grooves. It appears that use of recharge wells would require 50% less water than use of grooves while achieving the same reduction of drawdowns. In addition, it appears that for the same quantity of infiltration the damage when using recharge wells is less than in the case of infiltration grooves.

The initial investments in the case of recharge wells have been estimated at US$ 60 million for a recharge quantity of 40 million m^3/year (no damage is expected at all) and US$ 40 million for a recharge quantity of 20 million m^3/year (90% reduction of the total damage). The yearly operational costs amount to US$ 5 and 3.5 million respectively. The period allowed for depreciation has been set at 30 years.

As far as infiltration grooves are concerned, the cost of removal of the Holocene clay and peat deposits amounts to US$ 130 and 260 million depending on the above-mentioned hydraulic resistances (total damage will be reduced by 90%). The yearly operational costs are estimated at US$ 1 million.

By varying the recharge quantities and the dimensions of the infiltration groove, the costs of countermeasures can be weighed

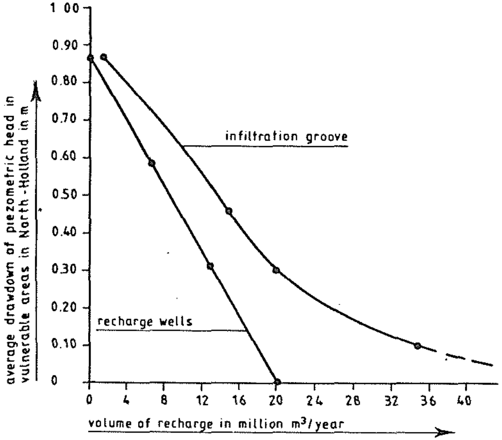


figure 10 Relation between the volume of artificial recharge (wells or grooves) and the average piezometric drawdown in the vulnerable urban areas.

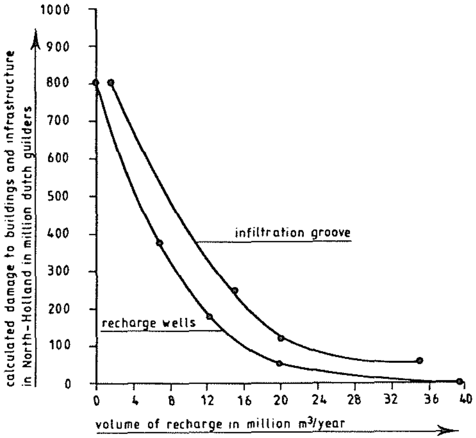


figure 11: Relation between the volume of artificial recharge (wells or grooves) and the damage to buildings and infrastructure.

against the remaining damage to buildings and infrastructure and the most favourable alternative can be chosen.

If drawdown of the piezometric levels in the upper Pleistocene aquifer is allowed to occur very slowly through phasing out the recharge well system over a period of about 30 years, it is to be expected that damage to buildings and infrastructure in the regions most vulnerable to damage will be greatly reduced.

If, on the other hand, this is done by phasing out the reclamation works (over five years instead of six months) damage to buildings and infrastructure will be reduced by approximately 50%. This will also mean an increase in reclamation costs.

4. GENERAL CONCLUSIONS

1. The ultimate drawdowns in the aquifers will under and around the polder have occurred 5 to 7 years after the start of the reclamation of the Markerwaard polder.
2. The maximum drawdown in the upper Pleistocene aquifer will be 0.25 to 1.25 metres in the area situated within a distance of 5 to 10 kilometres from the coast of North Holland.
3. These drawdowns can feasibly be countered by means of artificial recharge of water into the Pleistocene aquifers both by application of recharge wells and dredging infiltration grooves. Recharge wells cost less initially then infiltration grooves, but are more expensive to maintain than grooves. It seems possible that these countermeasures could gradually be phased out within circa 30 years.without significant damage.
4. A relationship was established between the extent of a countermeasure (quantity of injection water) and damage reduction, both for recharge wells and infiltration grooves.
5. A method has been developed to weigh the cost of countermeasures against damage to buildings and infrastructure, and thus allow the best solution to be chosen.
6. If it is decided to reclaim the Markerwaard polder, further research will be necessary on the operational and technical aspects of the countermeasures studied. Experiments on a semi-technical scale with recharge wells and infiltration grooves will have to be made on location.

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HIGHLIGHTS OF GROUND-WATER WITHDRAWAL SESSIONS

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The sessions devoted to the analysis of land subsidence due to ground-water withdrawal show that the land sinking linked to water pumping is still a problem of great interest and concern all over the world.

Williamson and Prudic presented a comprehensive review of the flow simulation and compaction in the regional aquifer system of the central valley of California, a case that has attracted a lot of attention in the recent past. The most significant results of the analysis demonstrated that over 13,000 sq km of the valley's area have subsided in the last 100 years due to ground-water withdrawals. Such an extensive subsidence resulted from the combination of the largest average annual ground-water pumpage in the world and a large aggregate thickness of compressible clay beds in the aquifer system. Ground-water flow and aquifer compaction were numerically simulated by a three-dimensional model. Most water usage was for irrigation. The volume of ground water in storage has been decreased by 74,000 hm^3 from the original conditions. It is interesting to observe that 67% was derived from the lowering of the water table, 28% from inelastic compaction and 5% from elastic storage.

Other cases of land subsidence due to ground-water withdrawals come from China. Guangxiao and Yiaoqi report on several examples of subsidence that have recently occurred in China. One of the most impressive is Shanghai city, which sank 2.63 meters from 1921 to 1965. The cause was subsurface water pumpage from fine-grained quaternary sediments in a confined aquifer system. Land sinking also takes place in mountainous areas, particularly in the shallow buried Karst depressions. The well-known mechanism of subsidence that occurred in the area of Shanghai has been reviewed by He-yuan, whose paper also provides some data on the compression of single layers.

Qingzhi and Xioujun analyzed the cause of land subsidence in the Tianjin area, which is located in the northern China Plain. There, the unconsolidated Cenozoic fine-grained sediments have a great thickness and, starting from 1950, a maximum cumulative amount of settlement of 2.15 meters has been observed. Again the main cause is overpumping from confined aquifers.

Land subsidence in northeastern Japan has been described by Matsuoka. The land lowering resulted here from withdrawals in deep water wells for agricultural irrigation purposes. Surveys from 1974 to 1982 revealed subsidence of as much as 28 cm. Expansion of urban areas has increased demand for water and decreased the recharge area. A remedial step is now being taken to bring in an alternative source of irrigation water from the Mogami River, a major river in the area.

Subsidence has been experienced also in Bangkok as discussed in the paper by Bergado et al. Ground water is pumped from unconsolidated deposits of sand, gravel and clay for industrial and urban needs. Starting in

1955 the water level decline has been 50 meters in 28 years and as much as 88 cm of subsidence between 1978 and 1983 in the east-southeast of Bangkok. Flooding of the city has therefore become a frequent problem.

Several types of land-surface sinking have been described in the Netherlands by Rietveld in a paper which also describes the method of hydrostatic levelling by means of a long lead tube filled with water.

In conclusion, subsidence problems resulting from large water withdrawals are likely to increase in the future in order to meet the necessities of developing industrial areas and increasing population in many parts of the world. By providing complete information on potential land subsidence to planners of industrial, urban, and other developments, it would be possible in some places to give support to the management and to thus avoid detrimental effects on the environment involved in the processes.

HIGHLIGHTS OF INSTRUMENTATION AND MEASUREMENTS SESSIONS

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This session comprised some excellent reports dealing both with classical methods, such as benchmark levelling, and more sophisticated methods like acoustic emission monitoring of land subsidence. The basic problem is that measurements are contaminated by "noise" which has to be filtered as much as possible. Some interesting papers on noise filtering were presented by Riley (Denver, CO, USA) and Cunietti et al (Italy). Riley reports on the frictional deadband of extensometers and noise due to thermal or mechanical disturbances originating near the land surface. Cunietti et al reported on a new type of rod-like bench mark used for the investigation of the stability of levelling bench marks in order to distinguish local disturbances (vibrations, mechanical or hydrostatic causes, oxidation processes) from deeper causes of land subsidence. However, acoustic emissions generated by deforming soil or rock masses were recently applied to subsidence monitoring in a coal mining area as reported by Koerner (Philadelphia, PA, USA). Acoustic events are analyzed for arrival times, converted to distances and then source located. Statton et al (Clifton, NJ, USA) measured microseismicity accompanying deep well fluid withdrawal. A microseismic monitoring network provided baseline data prior to the reservoir testing. An interesting case history was described by Pieri and Russo (Bologna, Italy).

The conclusion can be reached that measurements of land subsidence are of great importance. First of all they give information on what is happening at land surface but they also give information for the calibration and verification of mathematical models, which should be able to predict subsidence accurately.

HIGHLIGHTS OF HYDROCOMPACTION AND ORGANIC DEPOSITS SESSION

John C. Stephens
Session Chairman
Consultant
Fort Lauderdale, Florida, USA

The speakers covered a wide variety of studies on hydrocompaction and subsidence of organic soils that range from highly organic Histosols through the less carbonaceous Gleysols, and under land use conditions that vary from urban to agricultural. The subsidence of such soils is found to be caused by lowering the shallow water table, and land sinking begins soon after drainage and soil aeration.

The main reason organic soil subsides is due to two different actions:- (a) densification and (b) volatilization. Soil densification after drainage is due to several processes: shrinkage by drying; by loss of the buoyant force of ground water; by loading; and by compaction, usually by tillage. Such losses decrease soil volume only with maximum rates occurring soon after drainage. De Glopper's studies in the recently drained Netherlands polders showed the highest subsidence (1983) was approximately 90 cm in Gleysol underlain by 300 m of clayey-sand, and subsidence rates decreased as a function of log-time.

Organic soil volatilization is the result of changing the upper soil profile from a zone of saturation to a zone of aeration by drainage. Microbic organisms, which flourish under aerobic conditions, then attack the organic matter for use as a source of energy. By a complex metabolic process, carbon is converted to CO_2 gas and nitrogen is mineralized, which can be used as plant food, or if unused, may be hydrolyzed and flushed away by leaching.

Eggelsmann's research on peatlands in Germany confirms that Histosol subsidence rates generally increase with increased organic content, increased depth of drainage, and increased soil temperature, all of which increase the soil's microbiologic population. Contrarywise, subsidence is decreased by increased soil pH that inhibits microbiologic growth. Eggelsmann uses the ratio of annual precipitation to annual temperature to evaluate the climatic influence on subsidence.

Levin and Shoham's studies of organic soil reclamation in Israel show the same subsidence rate characteristics as cited above. They measured an average subsidence rate of approximately 8 cm/yr. Soil nitrate concentrations were as high as 360 μg/g. These researchers recommended keeping drained Histosols covered with perennial grass or crops to minimize ground--water contamination by nitrates. An important finding in the paper by Levanon and Levin was that alfalfa root systems release toxins that inhibit the growth of soil fungi, which in turn reduces the rate of subsidence. These findings should certainly be followed up to determine the agents involved and ultimate cause. Such information might lead to a breakthrough that could prove invaluable in conserving organic soils.

Snowden's data from New Orleans, Louisiana, USA, show the presence of surface or interstratified peats raise building development costs appreciably. Subsidence under pumped drainage is cited as the culprit. Buildings must be completely supported on piling. Grounds sink away from houses and must be raised by fill periodically. Unsupported streets, driveways, and sidewalks are a problem because of uneven settlement. All gas, water, and sewer pipes must be provided with flexible joints to prevent rupture.

In summary, organic soil subsidence cannot be prevented whenever a zone of aeration is created by drainage; however, the deleterious effects can best be minimized by keeping the ground-water table as high as crop and other land-use conditions permit.

HIGHLIGHTS OF OIL, GAS, AND MINERAL EXTRACTION SESSIONS

Francesco Marabini
Session Chairman
Institute of Marine Geology-CNR
Bologna, Italy

Giuseppe Ricceri
Session Chairman
Istituto di Costruzioni Marittime e Geotec.
University of Padova
Padova, Italy

A brief summary of the main points of the various papers in this session is presented. Geri, Marson, Rossi, and Toro in their paper have analyzed, from a topographic point of view, the subsidence of the "Travale" geothermal field. This area has a radius of about 2 km where producing and reinjection wells are present. In the central part of the area, during the period from 1973 to 1983, an average subsidence rate of 20 mm/per year has been measured. Since 1978, a series of gravity benchmarks has been installed in the area. The research tries to establish a correlation between geothermal field production and change of the gravity field.

The evaluation of permeability and compressibility of natural sedimentary soil of the Po Valley is considered in the paper of Brighenti and Fabbri. These parameters are of the uppermost importance in assessing the subsidence due to production from hydrocarbon reservoirs and are always required in numerical simulation models. Since the subsidence depends on several factors, the authors point out that it is sometimes very complicated to discriminate the various factors and quantify the parameters only on the basis of measurements of land subsidence.

Pöttgens gives a review of the ground movements in the Netherlands due to various mining activity. The older salt and coal mines are discussed, as well as the more recent gas reservoir. The author points out that different techniques must be used in each case in order to analyze properly the effects of various extractions.

Saxena and Singh present the state of the art of the subsidence research in India, associated with coal mine activity. Intensive research programs started especially after the first years of the seventies when the nationalization of the coal mines took place. So far, the findings of the first research have been helpful in extracting more than six million tons of coal located at shallow depth under surface structures. The authors also give equations to estimate the surface of soil profiles as a function of the maximum subsidence and of the length and shape of the flank of a profile.

Joshi in his paper presents an overall view of the problems connected with the presence of old mine workings in Southern Alberta, Canada. First of all he points out the actual difficulty in building up a complete picture of the underground situation because records of the mines are incomplete. However, in most mines, it is known that the room and pillar method has been utilized. Ground-water flooding causes floor heaving and roof sagging during mining activity and probably today the boundary soil is in a softened condition. With reference to the city of Lethbridge, situated upon the boundaries of these mines, the author concludes that this area will suffer in the future only a moderate subsidence and that light buildings can settle suitably.

During the 19th Century Industrial Revolution, deep utilization of natural resources took place in England. <u>Brook and Cole</u> present a series of studies that investigate the problems connected with the presence in the West Midland of a large number of abandoned mines. The authors point out that the detailed records of some mines are not readily available and that the presence of some of them is unknown. In the paper, aspects have been discussed that deal with field observation, engineering theory to analyze the subsidence, socio-economical and legal problems, availability of data and discovery of unsuspected mines.

<u>Okonkwo</u> examines the philosophy of mining activities and points out the price that each country has to pay in this field. The author goes into an extensive examination of subsidence in Nigeria's metal and coal mines. He reviews the role of underground mining operations in relation to the depth morphology of the various types of mines. For the metal mines, the mining method produces minimal and insignificant subsidence in Nigeria. The coal mines, because of their very favorable location in the Udi Hills, did not produce destructive effects to the surface.

In summary, today we can observe that several areas and fields of mining activities have been abandoned or are on the way to be abandoned. It is well known that subsidence will take place over these sites and that will depend on the depth, on soil and rock properties, on the type of mining activities and on the time since the mining activity has been abandoned.

Even if one can estimate the effects of subsidence by use of various techniques and approaches, it must be pointed out that in many cases there is a lack of data, mining records, and other pertinent information. Obviously the problems are more serious when there are urban areas in the vicinity and, of course, this case will be more frequent in countries where industrialization has required intensive mining activities.

CLOSING REMARKS

Lucio Ubertini
Institute of Hydrologic Research
National Research Council
Perugia, Italy

Italian National Representative, IAHS

Ladies and gentlemen, we are now at the end of the week's work. The time at our disposal has been employed in an excellent way with the illustration of very interesting and authoritative papers. Nor have occasions been lacking in which to enjoy the beauty of the city that has hosted us.

Nearly all the papers presented here concentrated on the concrete aspects of situations in which it was necessary to combat subsidence phenomena. Some of these have contributed towards a better and deeper knowledge of the phenomena in question, while others have dealt with means and methods of intervention to attenuate or eliminate subsidence.

This symposium has stressed how surveys in this field may be undertaken only if considerable human and financial resources are available, as the study of subsidence is an interdisciplinary task. Hydrogeologists, geologic and hydraulic engineers, geologists, mathematicians, statisticians, hydrometeorologists, and geophysicists all collaborate, and each one makes his or her own specific contribution.

Along with classical laboratory research go field experiments which may be difficult, expensive and, as may be seen in more than one case, original. I think this is the most effective direction to take if valid results are to be attained. Technology helps us by providing new equipment and the scientific community must be ready to utilize all available means, and, as well, must possess adequate resources for real commitment.

The commitment that the CNR researchers of Venice, along with other colleagues, have shown in studying the Venetian environment is worthy of the highest praise. It is necessary to do the utmost to encourage, help and potentiate our research workers who have given so much of themselves in their contributions towards helping Venice.

We hope that this symposium, which has provided an effective occasion for sharing experiences derived from studies carried out all over the world, will result in greater attention being brought to bear on the safeguarding of Venice and of other, similar, well-known cases by those who have been entrusted with this responsibility.

As far as we in Italy are concerned, I can assure you that our researchers' commitment will continue to be as incisive as in the past. With this symposium they have wanted to demonstrate the strength of their presence and they will endeavour to maintain a foremost position in working to protect Venice and other situations threatened by nature, by man, or by both.

Thank you for your effective participation in this symposium.

TISON AWARD

Following the presentation by the Exeter Assembly Organizing Committee of the sum of $13 000 to the Association and the acceptance of the idea of an annual prize to recognize the scientific contributions of young hydrologists to IAHS, the Bureau established the Tison Fund. Investment income from the Fund will be used to provide an annual prize of $750 according to the terms of the Award set out below:

TISON AWARD – RULES

1. The IAHS Tison Award aims to promote excellence in research by young hydrologists. The award will be announced annually and will be presented in a public ceremony during either an IUGG/IAHS General Assembly or an IAHS Scientific Assembly.
2. The Tison Award will be granted for an outstanding paper published by IAHS in a period of two years previous to the deadline for nominations. Nominations should be received by the Secretary General of IAHS not later than 31 December each year. The award will be announced by 31 May of the following year.
3. Candidates for the award must be under 41 years of age at the time their paper was published.
4. The Award will consist of a citation in the name of L.J. Tison and an amount of US$750. (If the successful paper is jointly authored, the monetary award will be divided equally between the authors.)
5. Nominations for the Tison Award may be submitted by the National Committees of IAHS and also by any individual or group of persons. They should be sent directly to the Secretary General of IAHS and should contain a reasoned argumentation.
6. The award decision will be made by a committee of seven members, one from each of the IAHS Commissions and Committee. The members of the Award Committee will be hydrologists of outstanding research reputation. The IAHS Bureau will appoint the members of the Award Committee, membership lasting for a period of two years. The Chairman of the Award Committee will be rotated among the different representatives of the IAHS Commissions and Committee.
7. The Award Committee may not recommend an award in any one year if none of the papers submitted is of sufficiently high standard.

ADDRESS FOR NOMINATIONS: Dr. J.C. Rodda, Secretary General IAHS, Institute of Hydrology, Wallingford, Oxfordshire OX10 8BB, UK.

INTERNATIONAL HYDROLOGY PRIZE

The General Assembly of IAHS held at Canberra in 1979 endorsed the principle of an International Hydrology Prize awarded annually on an individual basis in recognition of an outstanding contribution to the science. Nominations for the Prize are made by National Committees and forwarded to the Secretary General for consideration by the Nomination Committee which consists of the President, the First and Second Vice Presidents and representatives of UNESCO and WMO according to the following criteria:

- The International Prize in Hydrology shall be awarded to a person who has made an outstanding contribution to hydrology such as confers on the candidate universal recognition of his international stature.
- The contribution should have an identifiable international dimension extending beyond both the country of normal work and the specific field of interest of the candidate.
- The contribution may have been made through scientific work, as evidenced by the publication in international journals of scientific literature of a high standard, and/or through practical work, as evidenced by reports of the projects concerned. Preference should be given to candidates who have contributed through both scientific and practical work.
- The Prize may be awarded to hydrologists of long international standing or to those who, while having gained such standing only recently, exhibit the qualities of international leadership in the science and practice of hydrology.
- An active involvement in the work of IAHS and other international organizations in the field of hydrology should be counted as an advantage.

LIST OF PRIZE WINNERS

1981	Prof. L.J. Tison (Belgium)
1982	Mr. W.B. Langbein (USA) and Dr. V.I. Korzun (USSR)
1983	Prof. J.C.I. Dooge (Ireland)
1984	Prof. A. Volker (Netherlands)
1985	Dr. J.A. Rodier (France)